ON WATER TOWERS

By

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Thesis
[illegible]

CERTIFICATE

This is to certify that the work towards this thesis entitled 'ON WATER TOWERS', by Pradip Kumar Khan has been carried out under my guidance and it has not been submitted elsewhere for a degree.

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18/10/88

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Pradip Kumar Khan

October, 1988 Pradip Kumar Khan

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ABSTRACT

Design of structure is the process by which it is planned so that it will achieve its intended function in safety and at least cost. The advancement of technology has given mankind good structural designs in terms of complicacy & sense of beauty, and added a new term 'aesthetics' in engineering design. Water towers are no exception from that. The idea of the present work was conceived when 'LANDSCAPE DESIGNERS ASSOCIATES - N. DELHI' asked for structural design of a flat-bottomed water tower of 2000 Kl capacity supported on eight kinked columns & a central stem at a staging height of 20 m. After designing the structure parametric study of the same for staging have been carried out removing the central stem. The study of water tower design was continued to have a shape having least weight which will be transferred to the staging. A circular intze tank has been chosen for the basic shape and six other cases were considered by changing its various parameters viz., diameters and heights. It gives gave the idea the shape that would be available through the process of optimization.

DEVELOPMENT OF WATER RESOURCES SYSTEM

1.1 INTRODUCTION

In the beginning, people were not concerned with trying to keep water in any particular place. They merely built their various civilization where water was present. Their homes were first trees or caves and later small huts or shelters of varying design. When these were destroyed or damaged by high water, they quickly rebuilt or moved to higher ground. Since homes were simple in any event, people seldom experienced problems with wet or unstable foundations. When contingency occurred, it was easier to move than to solve the problem by ingenuity.

At first settlement were kept close to the water supply, so that their inhabitants could haul water by muscle power - animal or human - utilizing small containers. These served as civilizations first reservoirs. Seepage losses from these small vessels were only a minor problem, as replacement of water was close at hand. As civilization progressed people saw fit to place settlements or locations other than those blessed with water.

The exact time when humanity was first faced with the problem of moving water any appreciable distance in open to some debate. If we believe the writing of PLATO / 21 /, this may well have occurred some 12,500 years ago on Lost Islands of Atlantis. Of course to accept such an account , it is

necessary to believe the existence of such island. At any rate modern historians seemed agreed to one thing : humanity early became concerned with water conveyance and storage because of agricultural needs. That this water was used for drinking to is rather obvious, but it was farming pursuits that started the first primitive undertaking in the field of hydraulics.

1.2 TYPES OF RESERVOIRS

Throughout history, humanity has been faced with the problem of building facilities for storing water and other liquids. As we learned more modern building methods, design changes and other alterations were made as new materials became available. Although initial construction efforts were primitive by today's standard many structures built by ancients are still standing, and it is interesting to note that three out of four basic designs used today for water storage were developed thousands of years ago. Of the type listed below, only steel tank is a relatively modern development. The introduction of reinforcement steel into concrete, however did not change the basic concept of concrete tank : in merely improved the design.

Despite many variations, only two general methods are used in designing large water holding facilities. As the following tabulation shows, there are four variation of tanks, and the last of these may further be classified in three ways :

1. Cut-and-fill reservoirs
2. Tanks

A. Wooden

B. Steel

C. Masonry

D. Concrete

i) Cast

ii) Prestressed

iii) Wire wrapped.

1.3 CUT AND FILL RESERVOIRS

In this construction method, a depression with slopping side is created by digging and removing the earth with appropriate equipment. The earth so removed is utilized to construct fill or embankment around the top of the excavation to increase the effective height of the water and to reduce excavation costs. Part of the water is held below the original ground level (in cut) and part above this level (in fill). The slopes and the bottom of the excavation are then lined with appropriate material to prevent loss of water by seepage and also to protect the earthen component of the facility. Even when excavated material is not used in the actual construction, the designs is still referred to as cut-and-fill reservoir.

The most important advantage of cut-and-fill reservoir is its basic low cost. This is accompanied by a low profile,

an important consideration when the reservoir is to be constructed in a newly developing area when accent is on beauty. The structural strength of the facility comes from the compacted earth itself, which can be formed into place more economically than steel, concrete, or other materials of construction. No other type of reservoir can be built as economically as this one.

Unfortunately, this seemingly ideal water holder is not without its disadvantages. Although the earth is really holding the reservoir together, it must do so without fail. The technical aspects associated with this requirements are not assembled in any one place for easy access by the engineer. An engineer who has never built a cut-and-fill reservoir has considerable homework to do. Several disciplines are involved and the engineer must become familiar with all of them. In contrast, steel and concrete tanks are predesigned by various marketing organizations, and the engineer using the type of storage facility need only to copy this standard designs.

Moreover, unlike the steel, wooden or concrete tank, the cut-and-fill reservoir cannot hold water in vertical configuration. It must utilize the side supports that has slopes. A protective fence should surround the reservoir. Its specification are generally dictated by various agencies responsible for safety features of open storage facilities.

If the reservoir is holding potable water, either raw

or finished, additional safety features are required. Current practice is to build a fence on the outer embankment, so that the bottom of the fence is at least 8 ft. below the berm's top elevation. The fence is usually topped with various arrangements of barbed wire to further discourage entry.

1.4 TANKS

1.4.1 WOODEN TANKS

Wood is a very a popular material for use in water tank construction in early 1900s, and it continued well unto 20th century. At that time most of the water tanks used for railway water service were constructed of wood, and their use in this area did much to spur the growth of the wooden tank industry /21/. At present the rate of building new wooden tanks diminished considerably because of economic and technological disadvantages of the type of holding systems. Wood construction does offer some advantages for small tanks but the decreasing demand for such tanks has likewise been a factor in the decline of wooden tank business.

Seventy years ago, wood offered several advantages in tank construction. At that time, concrete and steel were not very common, there was a decided lack of technology in connection with their efficient use. In contrast wood wood working was a long-practiced art, and the joining and fitting of the staves were done with great skill. The staves were produced in the factory and properly coded and numbered

for easy assembly in the field / 21 /.

Aside from a change in tensioning hoops from flat bands to round rods, little change in construction procedure has been made in the years. The staves varied in thickness unto about 3 inch., depending on the size of the tank. A groove was cut at bottom of the staves for receiving the edges of the planks that made up the bottom of the tank.

Hoop tension was achieved by tightening nuts on specially designed lugs. Tensioning could be increased at any time, merely by additional turning of the tightening nuts. Many early failures resulted when the originally designed flat tensioning flats were used. Corrosion of the hoops next to the tank went unnoticed until tank literally fell apart. A change to rods with circular cross section did much to eliminate this problem.

On the credit side of the ledger, wood is free from the rust problem that plagued steel construction at the time, and wooden tank has certain inherent protection against frost and cold weather, which cause problem to the concrete tank. When the fibers of the wood in the tank is thoroughly saturated decay is practically impossible. Also if the wood is perfectly dry, there is little chance for decay. It is the in-between condition that cause the wood to deteriorate. Even then the wooden tank has a respectable life span, especially if the better aging timbers are selected.

1.4.2 STEEL TANKS

Although historians disagree as to the exact time when iron was first smelted the consensus of opinion places at the event at about 1500 B.C. Despite this early beginning, things went slowly, as the technology of steel making proved to be very complex. In addition the massive equipment and enormous power is required to roll steel into plates were slow of attainment.

In short, the jump from iron to steel did not happen overnight. Again history is somewhat vague as to when this took place, and its occurrence is further being crowded by the complexity of the change. High-quality steel was produced in the ancient past, and in 384 B.C. Arristotle described its manufacture. The steel was made in India and the metal shipped to Damascus where it tempered and hammered into sword of "Damascus Steel" / 21 /. For steel to be useful in the fabrication of all-steel water tanks, it must be of plate form. This requires that the steel ingots to be rolled into plane sheets. Although evidence shows that hot rolling of steel took place in the late 16th century, the process was crude and yielded steel plate of very poor dimensional tolerances.

As the steel industry learned how to roll plates into thin sections, it began to look into potential business in the waterworks field. Quick inroads was made into this area because of apparent advantage of steel-type construction

over concrete and wood. No spalling, no cracking, and no problems at wall-floor junctions were of the selling points. The industry also boasted that the standard designs could be furnished to the engineer and this did an effect in the speeding up of the specification process. On first cost basis, steel tank seemed to offer decided advantages over its concrete competitor, particularly in the smaller sizes.

It soon became apparent, however, that the steel tank did not provide the low-cost carefree facility predicted by the seller and hoped for by the owner. Corrosion, then as today, was a very serious problem. It strikes on four major fronts: (1) where the steel is in contact with the water, (2) where the steel is in contact with the moist air above the waterline, (3) in the foundation areas where the steel touches the soil and, (4) in areas of contact of dissimilar metals.

1.3.3 MASONRY TANKS

The masonry tank for water storage was a popular form of construction at one time. In fact, the further back we go in history, the more popular it was / 21 /.

To compensate for their structural weaknesses, very early tanks built on the principle that any destructive force could be overcome if enough mass was designed into the wall. Such was sound engineering logic in ancient times when rulers had an abundant supply of low-cost slave labor. It is

true that the slaves had to be housed and fed, but these costs were kept to an absolute minimum. Such a system for funding was hard to beat. It eliminated the need for cost control and value engineering since no one could improve on the cost of slave labor.

From seepage control standpoint, too, masonry construction represented sound engineering. The building blocks were large and dense, and the resulting mortar path between them was a long one. Hence initial seepage was relatively low. Later, or sometimes immediately after construction, bitumen-type linings were applied to the inside of the tank. Actually, they served as primarily construction joint seals, rather than linings because of the impervious nature of the rocks used.

Today high labor costs and modern design has eliminated masonry tank construction except in very special cases. Masonry is difficult to reinforce, yet because of its poor structural stability such reinforcement is generally required in the construction practice. Early tanks of the present era were often built of rough stones, with even rougher texture mortar joints. This surface condition translates into an expensive cost of preparation of the wall to receive any future lining system. For this reason it is not economically feasible to line small masonry tank. Tanks that leak are usually abandoned or operated at reduced water levels.

In its limited applications today, masonry construction for water holding facilities generally makes use of cement or cinder blocks. This approach eliminated formwork and claimed to be a less expensive method of construction. Walls of limited height and are often extensions of existing facilities for the purpose of increasing their capacity. Some new facilities have been built using block construction to increase the holding capacity of cut-and-fill reservoir designs. In both cases reinforcement is used to attain necessary structural stability.

The block design mentioned above requires a lining of some type to keep seepage loss within respectable limits, since the blocks are quite porous. During construction extra care must be used to ensure that the mortar joints are given a smooth finish, flush with the inside face of the blocks. This is particularly important of thin membrane system are to be utilized as the lining.

With the advent of steel and modern concrete tanks, The masonry design rapidly declined in favor. Their main to claim to fame is their early origin. They also played an important role in transitional development patterns of the concrete tank.

1.3.4 CONCRETE TANKS

The concrete tank as we know it today was developed through a process of evolution. At one point in the process,

the development was actually reversed itself at the direction of the cut-and-fill reservoir. Early tanks made limited use of cement. The cement was made up into a mortar and used between stones to construct vertical tank wall. Thus the reservoir was built-up type of structure. As a finishing operation a thin plaster mix was sometimes troweled onto the resulting surface to give added watertightness and a more pleasing appearance. This type of construction did the intended job when seepage losses were tolerated as a necessary evil and when labor costs were low. Walls were very thick by today's standards, often exceeding 12 inch.; 2 ft. wall sections were not at all uncommon.

Early technology centered around the vertical wall structure that is referred to as tank. This is not surprising , since the above-grade tanks are cousins to cisterns and wells. The building of later structure is an ancient art, so the adoption of built up masonry construction to above-grade tanks was a logical step. It was also a step that could be taken piecemeal, in that portion of the tank could be set into the ground and the remainder extended into the air. As experience was gained, higher tanks could be constructed, thus decreasing unit storage costs.

The development of portland cement was closely associated with waterworks facilities. But the use of portland cement in waterworks facilities had a very shaky

beginning, because the original block fell into pieces when immersed in water. Troubles were due to improper mix and curing conditions, lack of proper reinforcement steel, design inadequacies, and a generally poor knowledge how concrete and its reinforcement react to various environments. With the advent of more effective communications and the development of trade associations many problems have been solved. It is still necessary, though, to conform to good design practice coupled with high-quality construction standards and to evaluate carefully all factors that influence the final overall design.

Before reinforcing steel was available or its use was well understood, the designer, if he or she went into air with the tank, could logically do so only by employing masonry tactics. To be on the safe side, original designs included heavy wall sections. This feature was copied later by most designers, as innovation was not a popular trend. Designers and builders stuck to things that had worked in the past.

The designers had the idea of placing retaining wall on the sloped earth, allowing the later to support the tank walls. In this approach the tank is no longer a tank, but becomes a reservoir. The "walls" were still brick and mortar, rock and mortar, or, later, unreinforced poured concrete. Combinations were also used, particularly ones utilizing a concrete overlay on top brick or rock

construction. Exactly when this construction principles were first utilized in modern times is unknown, but many facilities of this type were built in the years before 1920.

The concrete evaluation process, which seemed to culminate in the cut-and-fill reservoir, did not stop there. Instead it reversed itself and went back to the vertical direction whence it came. This happened for a number of reasons. Excavation methods were crude and costly. Trucks did not exist, nor did heavy earthmoving equipment. Also, though a less of a problem then than now, the cut-and-fill reservoir required considerably more ground space than did the tank. For a while, however, the later argument was counterbalanced by the fact that the building materials, construction techniques of the day, and psychological factors somewhat limited the height of the tank construction.

During this period of evolution, formed and poured concrete tanks made their appearance. Again walls were very thick. Even though reinforcements were introduced, it was crude in comparison to that used today. It went to both extremes, being either thin wire mesh or very heavy, twisted cross sections. The concrete quality varied greatly . Some of this structures are still in excellent condition, the concrete being hard and dense, whereas in others the concrete is of very poor quality, appeared to have suffered because various combinations of poor mixing, low cement

content, or chemical degradation processes. Cracks in the structures were common, this being a particularly serious difficulty with respect to cold pour joints. Builders were not familiar with the problem, and cement mixing which was done by hand, could not keep up with efficient pouring schedules.

Tanks were often set into the ground, or at least a major portion of their depth was in the ground. For this reason seepage went unnoticed or it was just that no one bothered with remedial measures. On some old tanks, though, evidence of crack repairs using concrete and asphaltic materials is visible.

There was a difference of opinion with respect to overall design of early concrete structures. One school of thought decreed that the structures should be built with vertical walls, whereas another believed that the sloping walls were better. The later group argued strongly. Formwork was eliminated, they said, and reinforcement was not needed. Although these things represented only a small money savings by today's standards, they still accounted for appreciable percentage of the total job cost. An equally enticing advantage of sloped wall construction was its apparent simplicity, both for the designer and the constructor. Of course, they were soon to find that these tanks, more designated as cut-and-fill reservoirs, present a batch of problems of their own. These center around the subject that how to keep the water inside the structure from leaking out

of it. Builders of vertical-wall tanks were well aware of leakage problems in their creations, and they seemed to find some comfort in the fact that they were not alone in this respect.

To cut the cost and improve the quality of concrete tanks, reinforcement steels were added. This reduced the concrete mass by absorbing and restricting tension loads. During this period, construction of concrete tanks evolved in two directions : (1) the reinforced tanks, and (2) the prestressed tanks.

DESIGN OF WATER TOWERS

2.1 INTRODUCTION

Water towers are usually constructed of in situ reinforced or prestressed concrete and generally consist of an elevated tank supported on columns or on a massive central stem or combination of both. The older, plain, square or rectangular concrete tanks on four columns have in recent years given place to very elegant and attractive structures. In some instances, particularly on the Continent and overseas, the tank is cast at ground level and then jacked up the central stem.

2.2 EXTERNAL APPEARANCES

Water towers are normally conspicuous structures as they are located on relatively high ground and in addition they are elevated.

The engineer concerned should cover the aesthetics of design of structures and some people will no doubt consider that this is the province of the architect rather than the engineer. It is suggested that good engineering should include a design which is aesthetically satisfying. The great engineers of the past have always achieved this. One of the many advantages of concrete as a construction material is the variety of external finishes which can be obtained.

When considered the type of finish, the problems of weather staining must be given close attention and careful detailing is needed for an otherwise pleasing structure is not to be spoilt by unsightly streaks and patches. There has been a move away from smooth, plain finishes to a rougher type of surface, such as a broad-marked, exposed aggregate and sculptured. When considering the type of finish required, it is important to remember that the mix design chosen must be suitable for watertightness as well external appearance.

2.3 WATERTIGHTNESS IN TANKS

Nothing spoils the appearance of an otherwise attractive water tower more than seepage of water through cracks, joints, and defective concrete. It is not simple matter to design and construct a large water tank at a height of 20-40m. above the ground, so that it is literally 'bottle' tight. A lower standard of watertightness can cause disappointment from an appearance point view.

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It is impossible to construct a structure of any size without joints and in a water tower these will be visible. Therefore they should be featured and form part of the external design, and must be completely watertight. With in situ reinforced concrete water lining should be given careful consideration.

The lining can be either performed material, such as PVC or polyisobutylene (PIB), or in situ coating based on polyurethane and epoxide rasins / 1 /. For towers containing potable water, the lining must be non-toxic and non-taining, and must not support the growth of bacteria, fungi and algae.

2.4 SKETCHES

A few sketches of water tanks are here shown here which are being used in practice / 1,2,3,6,17 /. At first full sketches are drawn & then sectional views has been shown here.

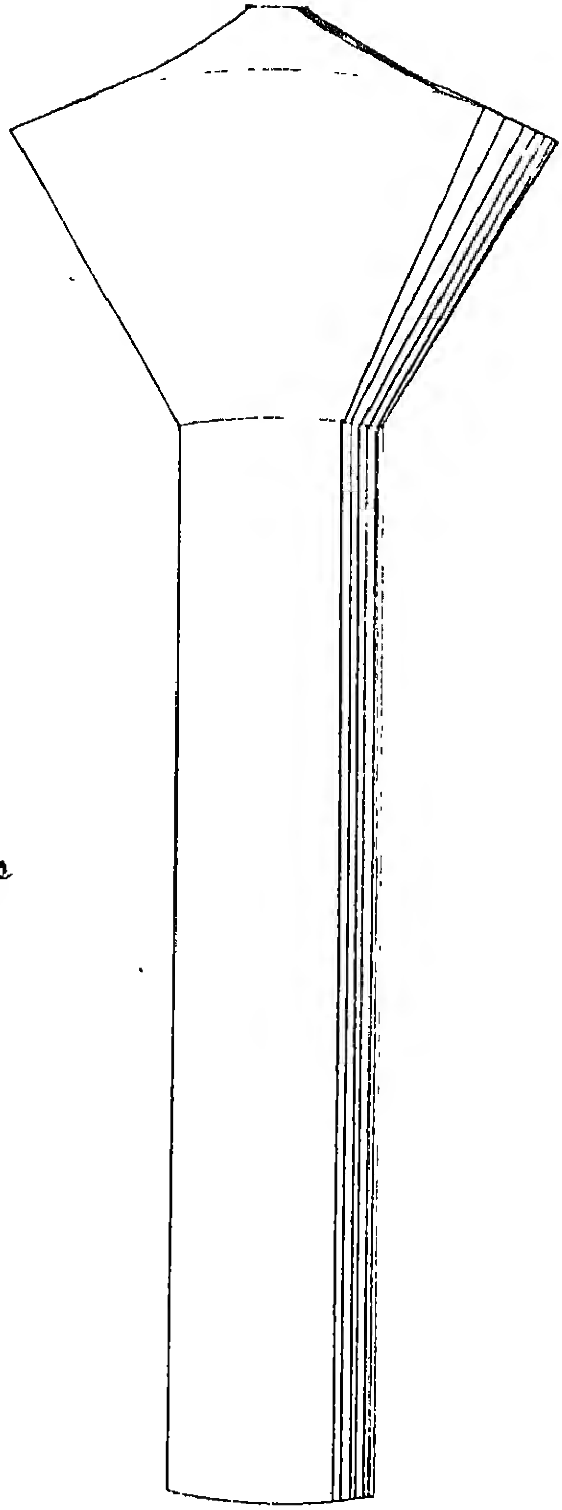

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Fig 1a. - Sketch of water tower

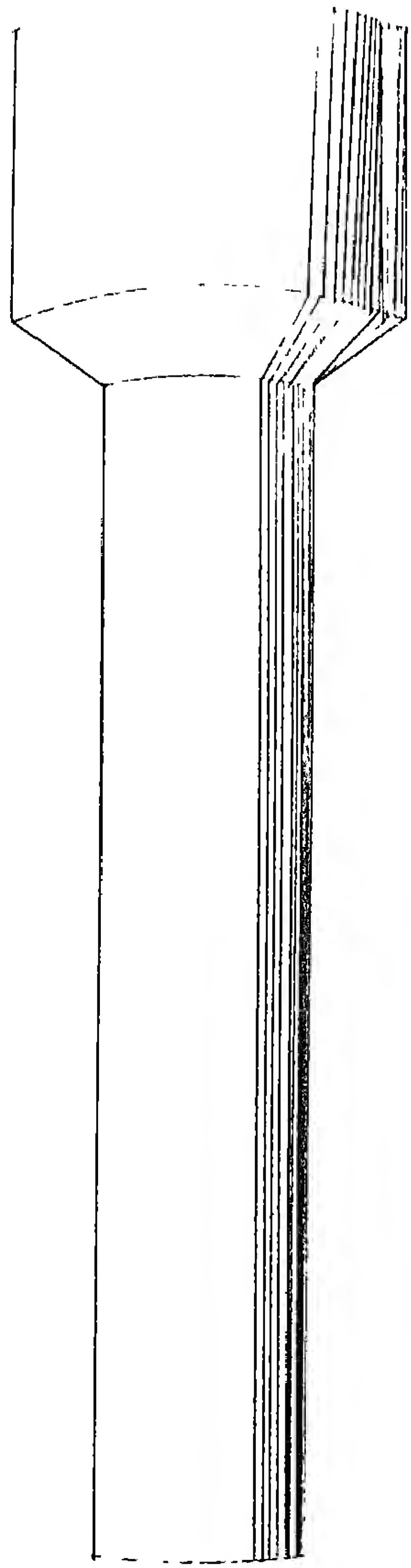


Fig. 1b. - SKETCH

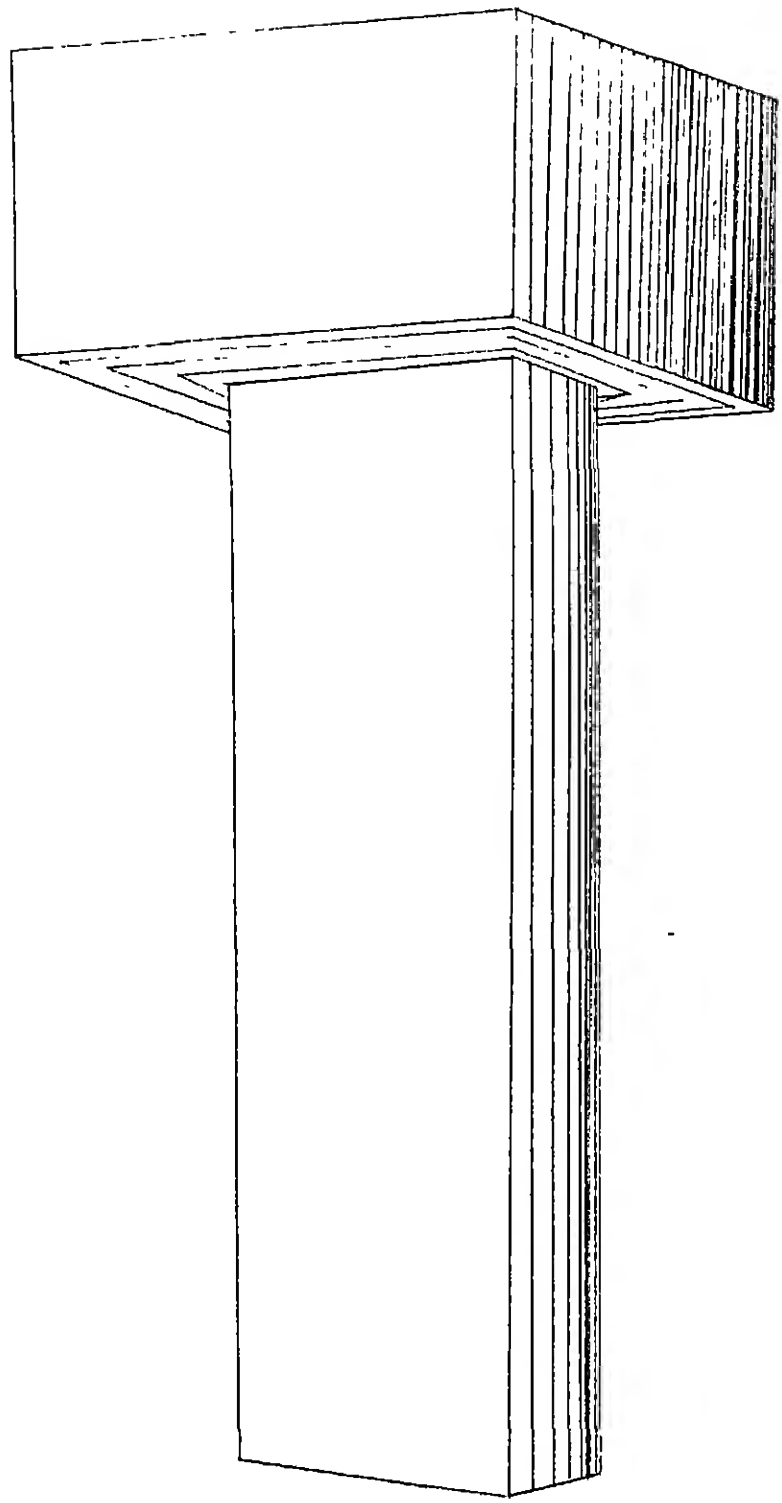


Fig-1C - SKETCH

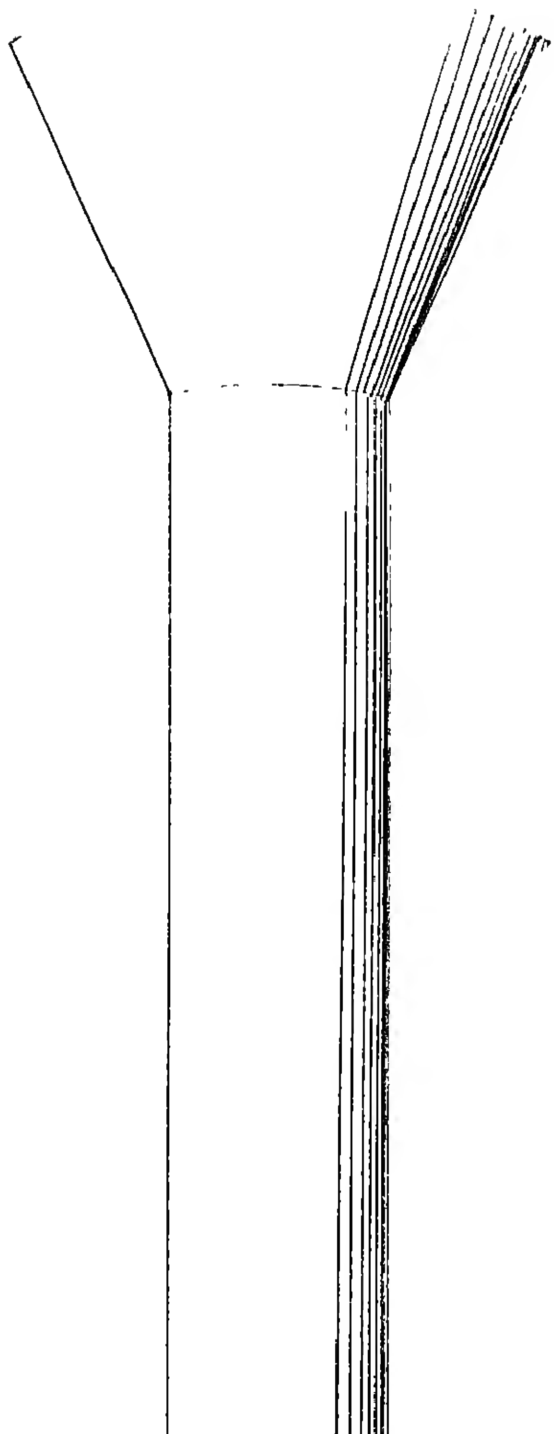


Fig.-1D- SKETCH

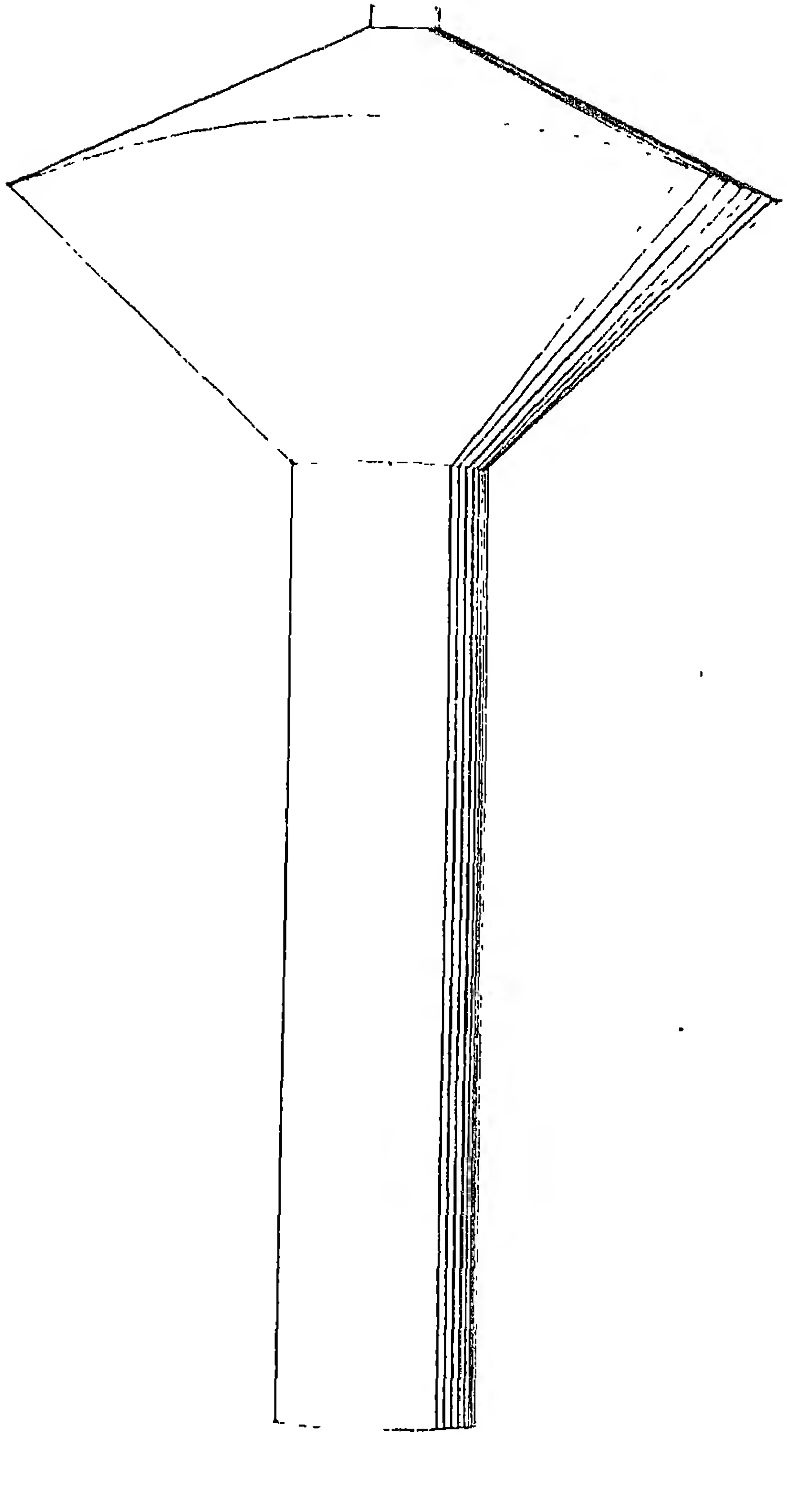


Fig - 1E - Sketch

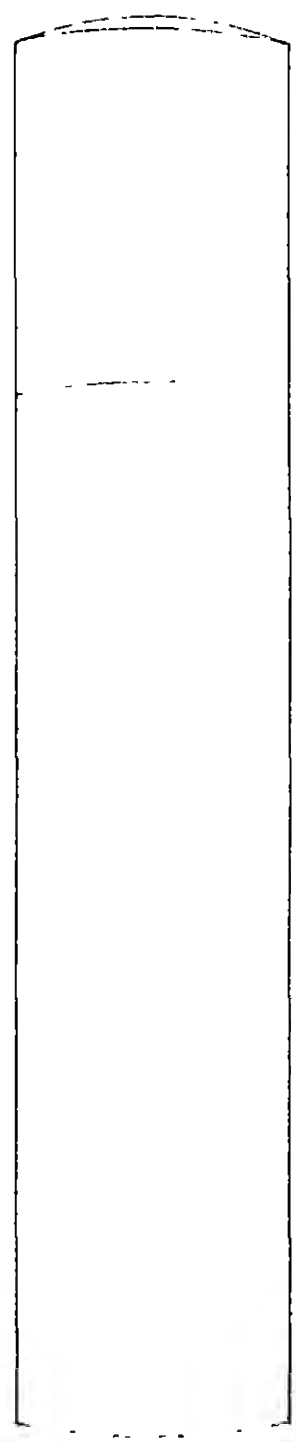


Fig- 1F - SKETCH

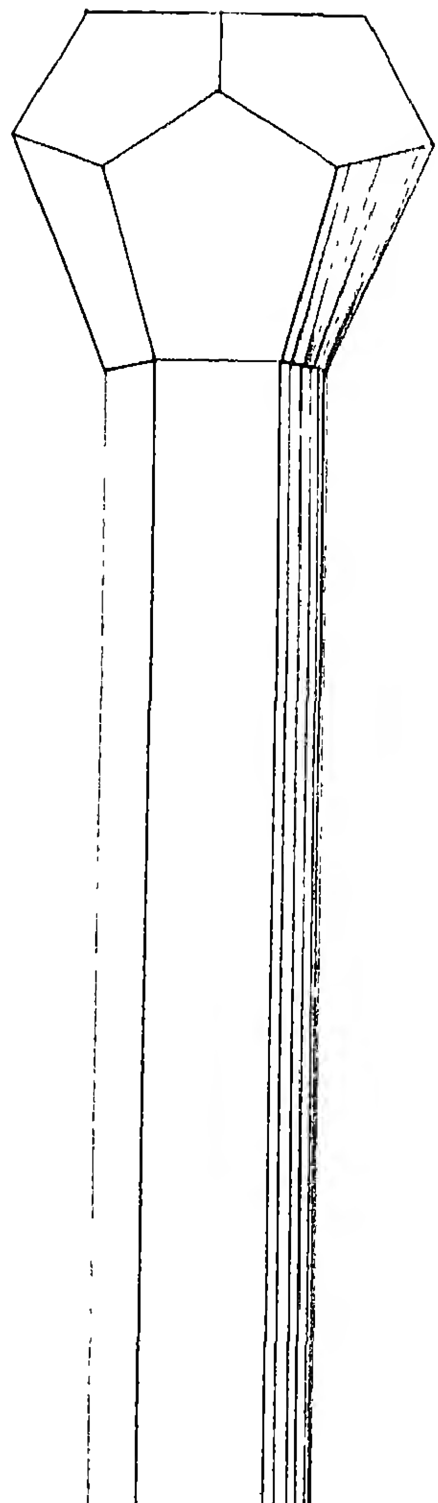


Fig.16 – SKETCH

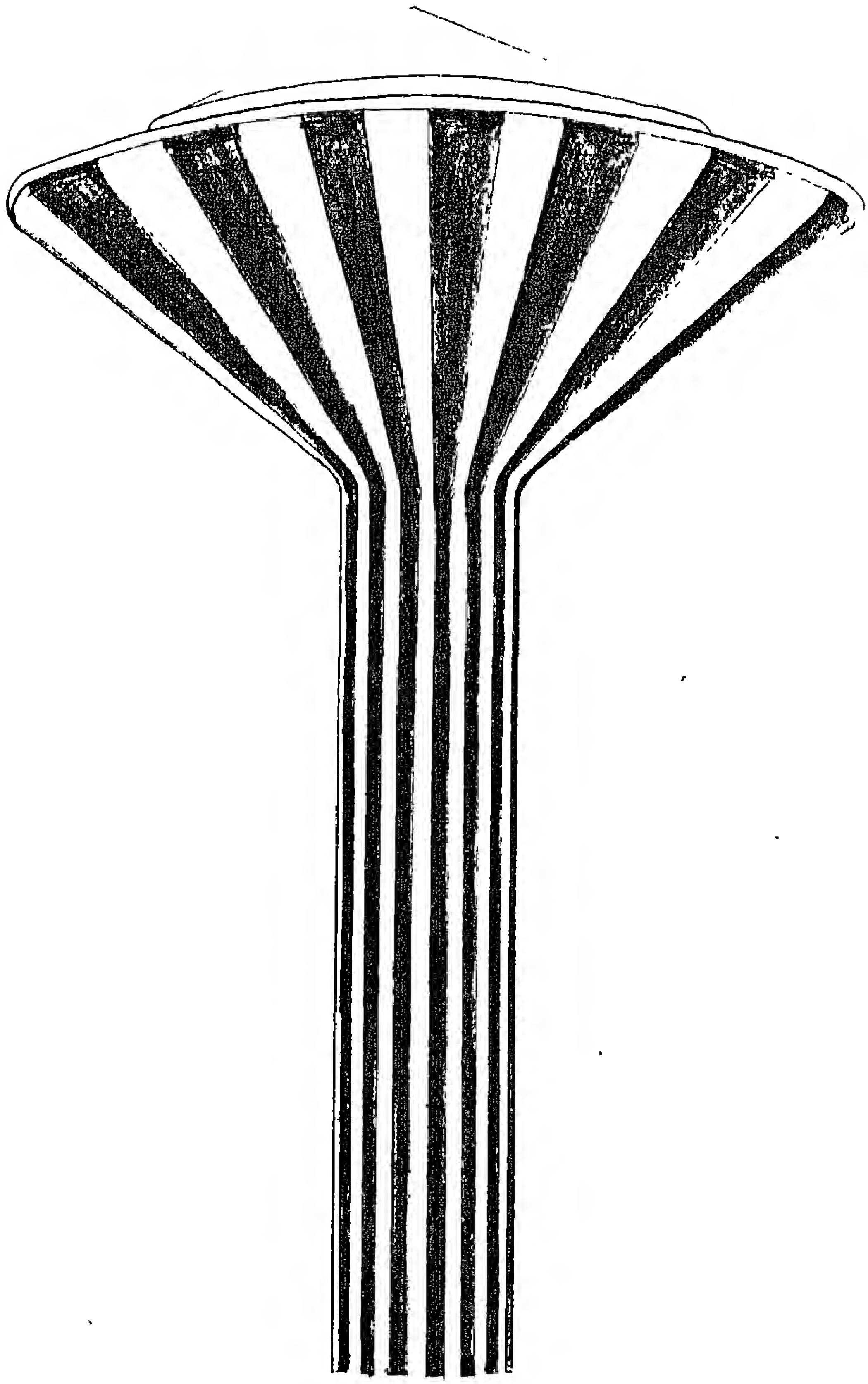


Fig. 1 H - SKETCH

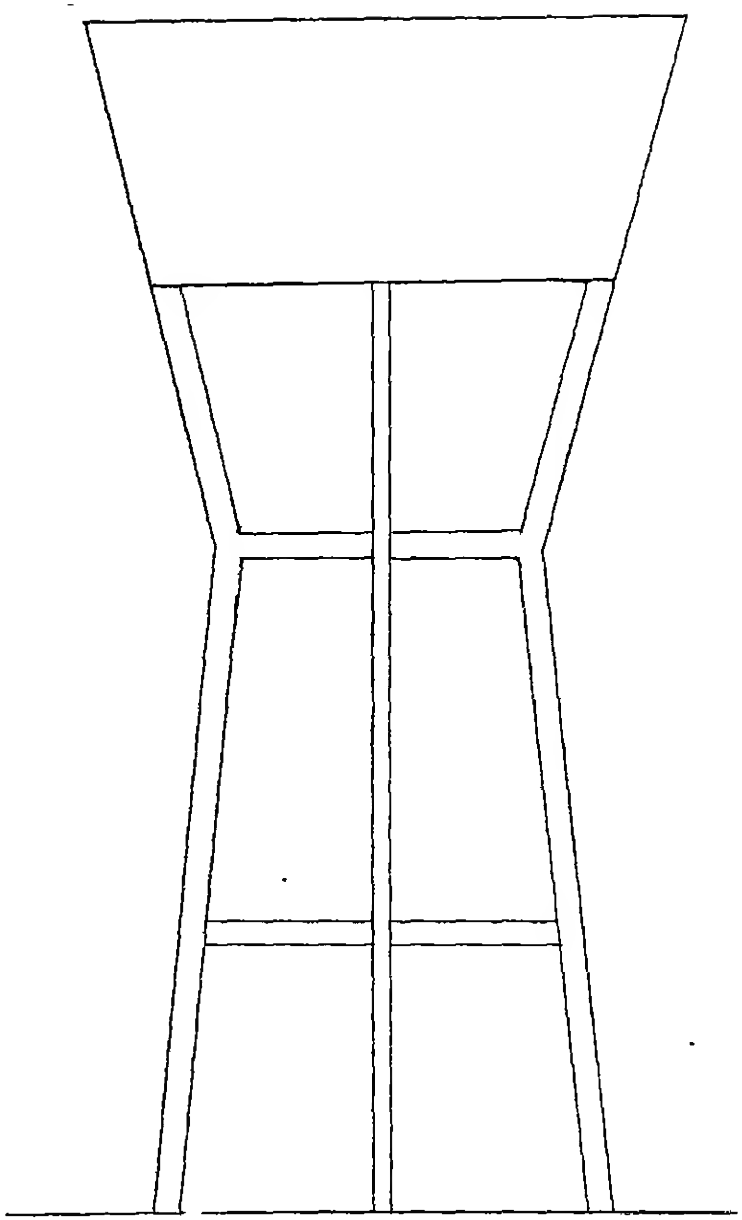


Fig.II- SKETCH

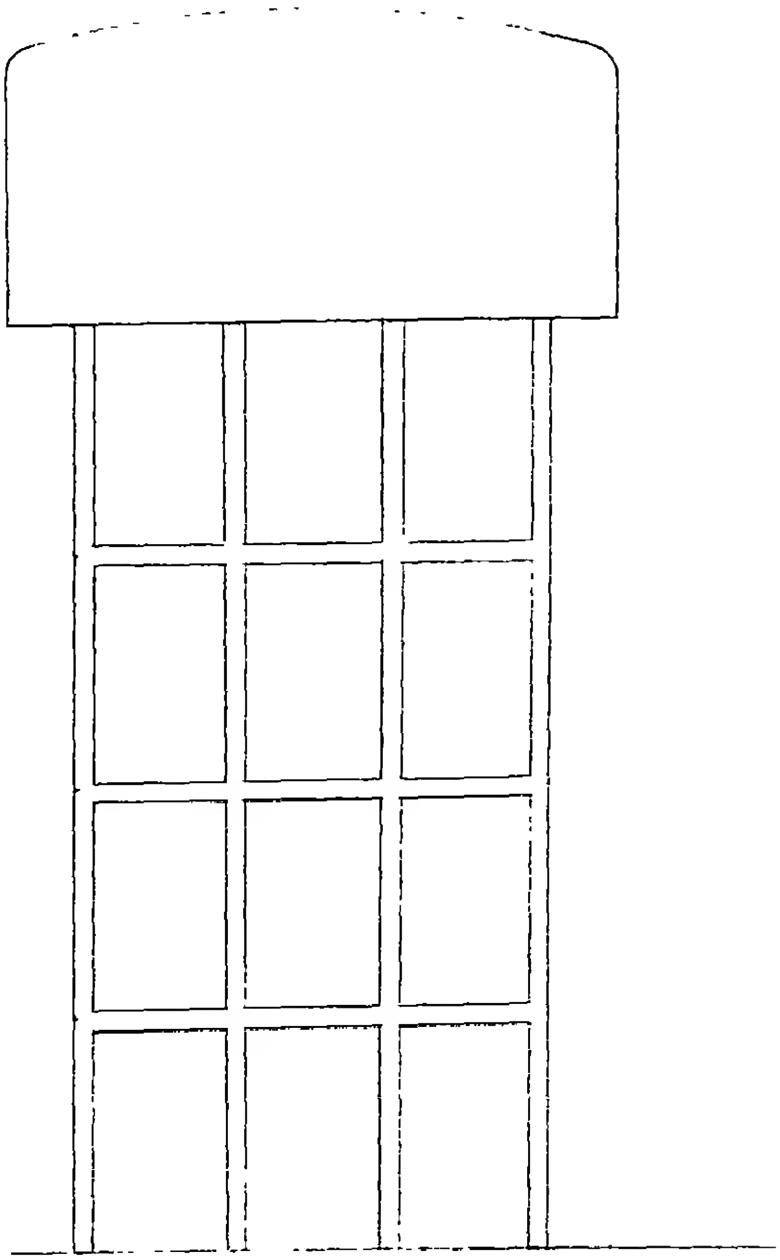


Fig.-IJ- SKETCH

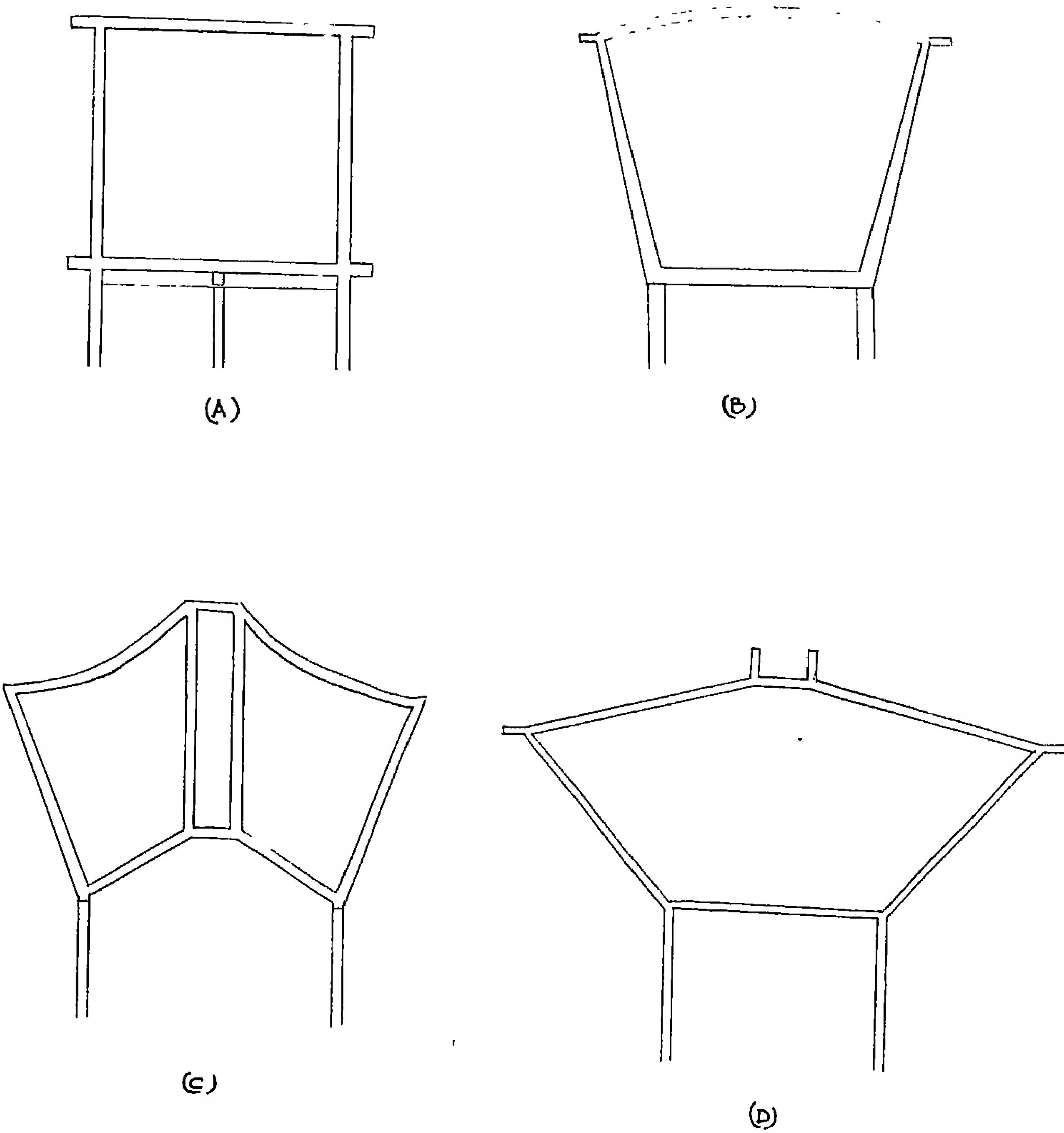
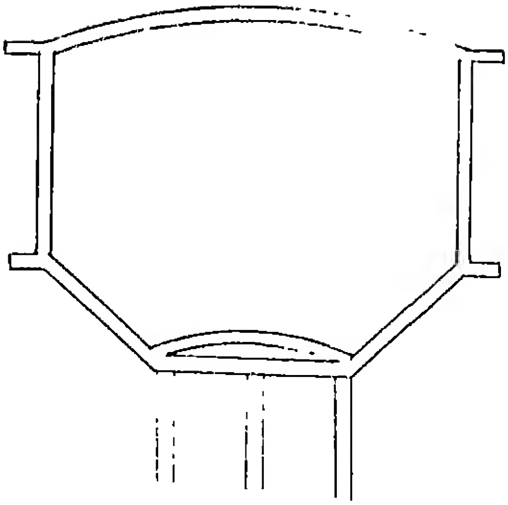
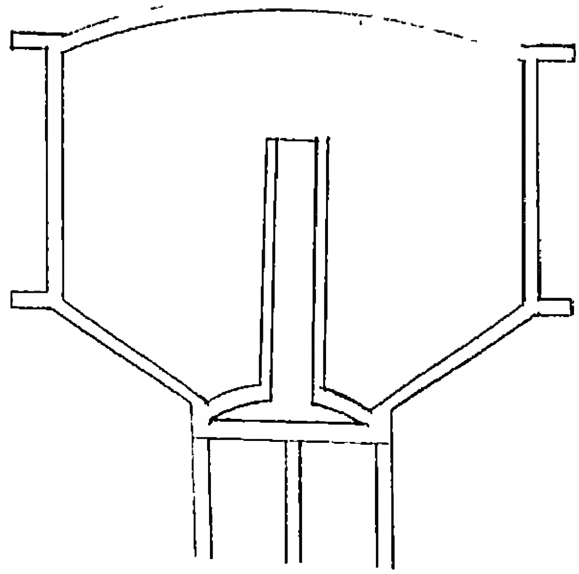


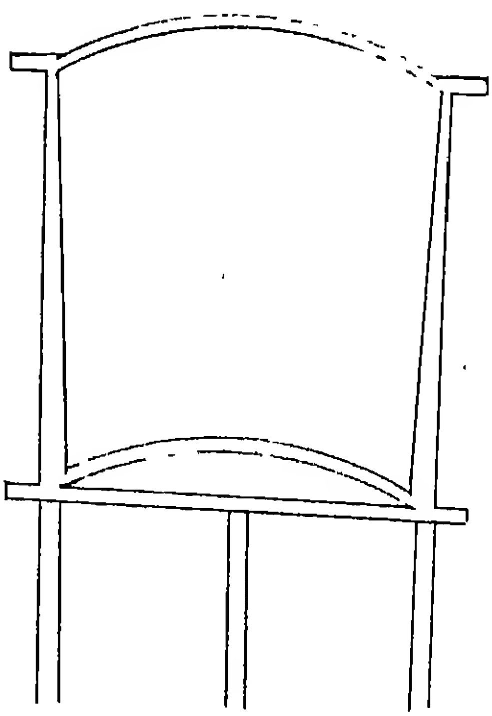
Fig 2. sections of watertower



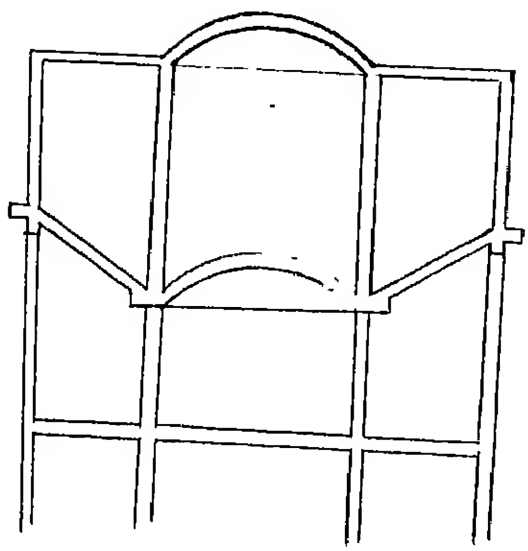
(E)



(F)



(G)



(H)

Fig 2- section of watertowers

3.1 Design of Water Tank ::LIST ::

3.1.1 SUBSTRUCTURE

A) Below Columns:F1

B) Below Shaft:F3

C) Connecting beam between column & shaft foundation:F2

3.1.2 SUPERSTRUCTURE

(Should state the ref. to Fig. etc. have Elevation, to make it)

A) Columns

- i) Level 1:CL1
- ii) Level 2:CL2
- iii) Level 3:CL3

B) Shaft:SH

C) Tank wall:TWL

D) Slabs

- i) Top part:SL1
- ii) Bottom part:SL2

E) Beams

- i) Top slab
  - a) Radial beams:RB1
  - b) Circuler outermost beams:CB1
  - c) Circuler intermediate - 1 beams:CB2
  - d) Circuler intermediate - 2 beams:CB3
  - e) Circuler innermost beams:CB4
- ii) Bottom slab
  - a) Radial beams:RB2
  - b) Circuler outermost beams:CB5
  - c) Circuler intermediate beams:CB6
  - d) Circuler innermost beams:CB7

F) Plate at midlanding:PLT

G) Staircase:STC

H) Dome at top:DME

3.2 DESIGN OF WATER TANK

DATA :

Capacity	=	2000 KL
Free board	=	0.30 m.
Staging height	=	20.0 m.
Shaft radius (outer)	=	4.00 m.
Shaft thickness	=	125 mm.

Inner diameter of water tank = 24.0 m.

Depth of water in tank $= 2000*4/\Pi*(24^2-4^2)$

$= 4.547$ m. $\simeq 4.6$ m.

Total height of container = 4.90 m.

Let depth of slab = 0.3 m

So self weight = 7.5 KN/m

Weight of water = 46.0 KN/m

Total weight = 53.5 KN/m

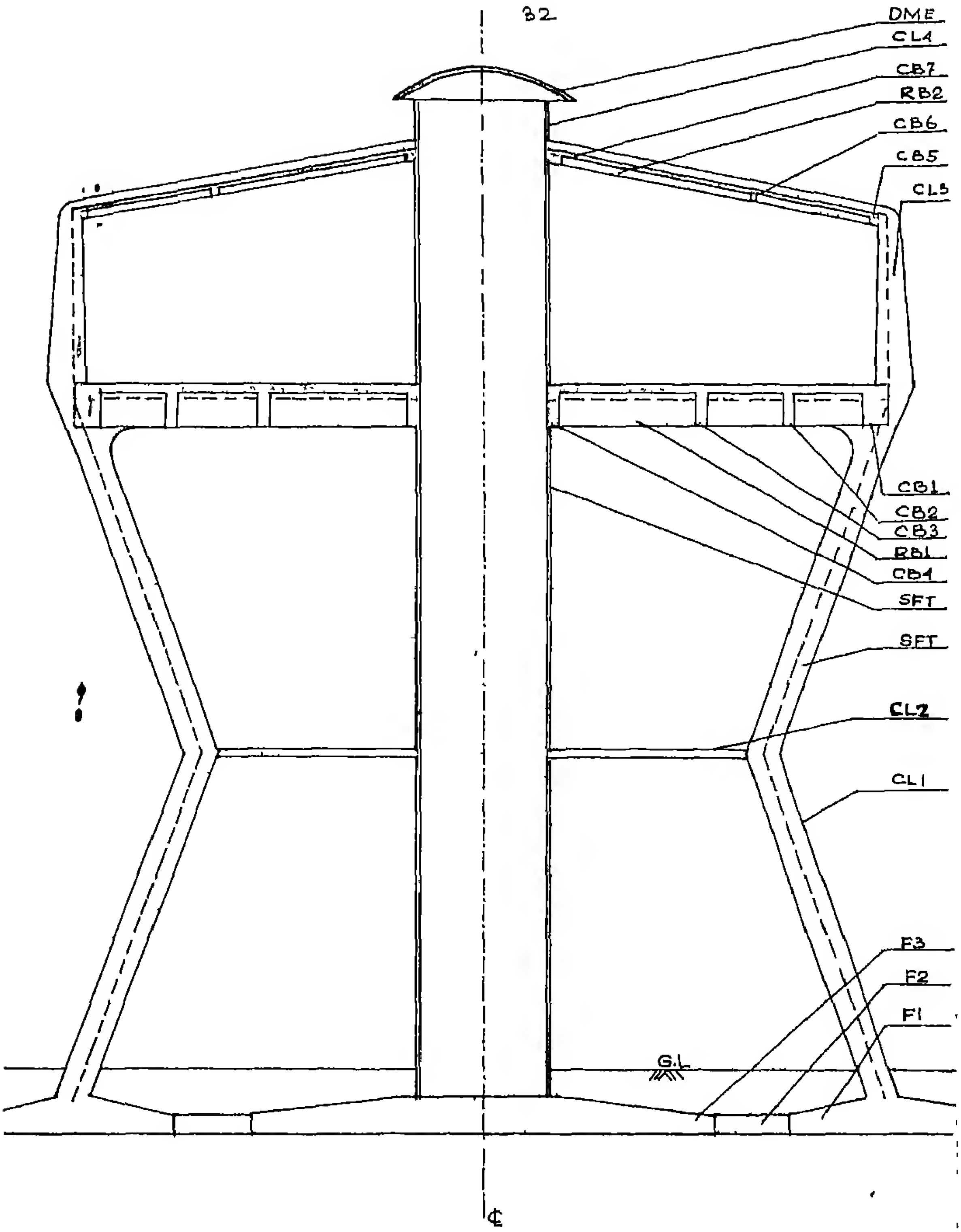


Fig -3 CROSS SECTION OF THE TANK

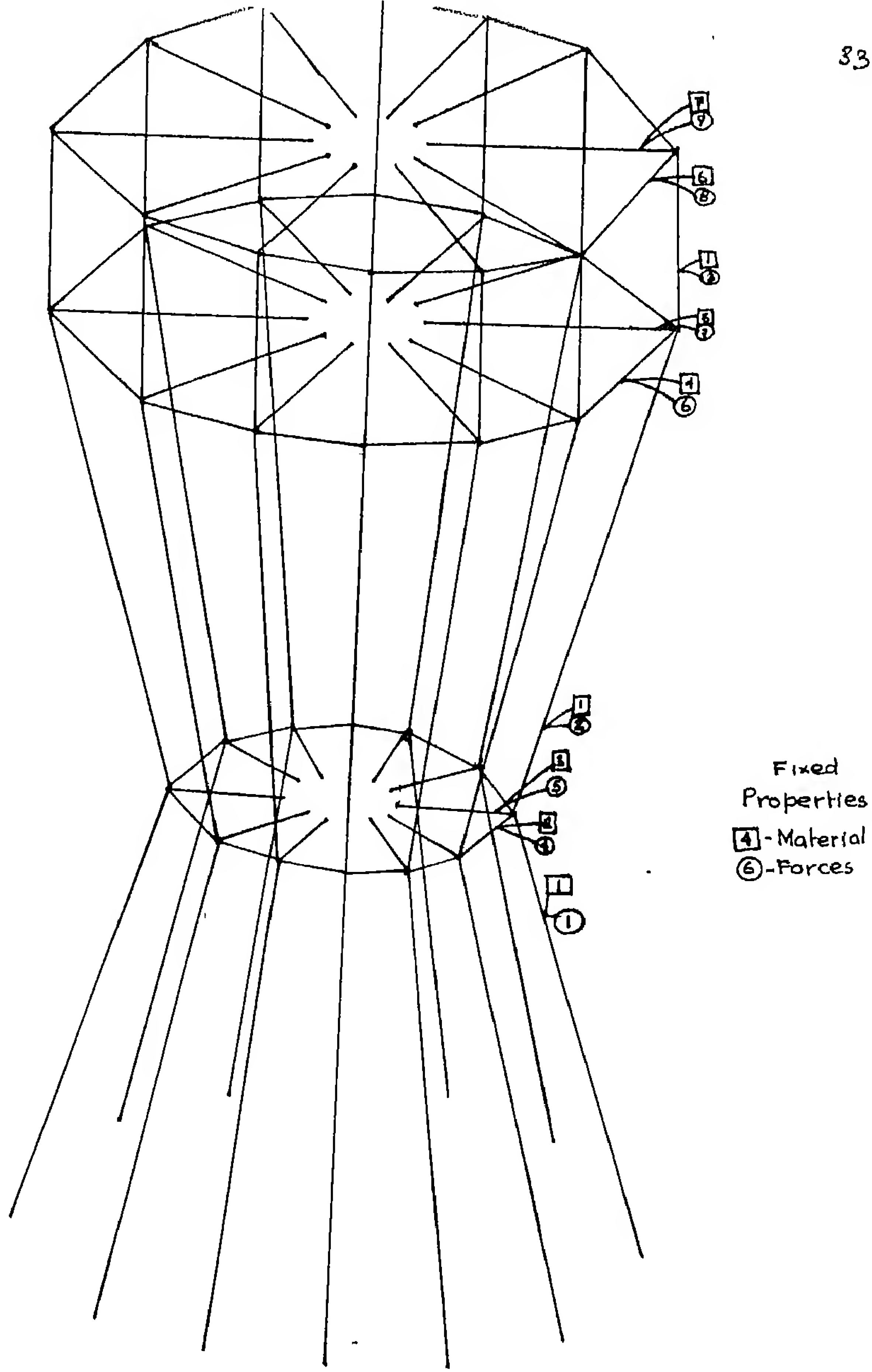


Fig.-4 IDEALIZED SECTION OF TANK FOR ANALYSIS

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3.5 STATIC ANALYSIS :: Fixed end force sets ::

No. : 1

$\theta = Tan^{-1}(10.0/3.5) = 70.710$, $\omega = 1*0.5*25 = 12.5$ KN/m
$\omega_d = \omega/Cos\theta = 37.84$ KN/m, $l = 3.675$ m
$M = w_d l^2/12 = 42.586$ KNm, $\omega_2 = 22.969$ KN
$\omega_\alpha = \omega Tan\theta = 35.71$ KN, $\omega_1 = \omega_\alpha l/2 = 65.625$ KN

No. : 2

$\theta = Tan^{-1}(10.8/3.78) = 70.71^o$, $\omega = 1*0.5*25 = 12.5$ KN/m
$\omega_d = \omega/Cos\theta = 37.84$ KN/m, $l = 3.78$ m
$M = \omega_d l^2/12 = 45.054$ KNm, $\omega_2 = 23.625$ KN
$\omega_\alpha = \omega Tan\theta = 35.71$ KN, $\omega_1 = \omega_\alpha l/2 = 67.857$ KN

No. : 3

$l = 5.65$ m, $a = 0.50$ m, $b = 0.55$ m, $s = 4.60$ m
ω = Triangular water pressure of height 4.60 m.
$M_i = 441.557$ KNm, $M_j = 285.344$ KNm
$R_i = 425.513$ KN , $R_j = 239.227$ KN

No. : 4

$l = 4.451$ m, $\omega = 0.6*0.4*25 = 6.0$ KN/m
$M_i = -M_j = 9.904$ KNm, $R_i = -R_j = 13.352$ KN

No. : 5

$l = 6.5$ m, $\omega = 0.6*0.5*25 = 7.5$ KN/m
$M_i = -M_j = 30.14$ KNm, $R_i = -R_j = 24.375$ KN

No. : 6

$l = 6.178$ m, $\omega_{self} = 32.5$ KN/m, $\omega_{wall} = 18.375$ Kn/m
$\omega_{slab} = 9.0$ KN/m, $\omega_{water} = 46.0$ KN/m (Trapezoidal)
$M_i = -M_j = 344.853$ KNm, $R_i = -R_j = 309.59$ KN

No. : 7

$l = 10.0$ m, $\omega_{self} = 12.81$ KN/m (u.d.l.)
$\omega_{water+slab} = 336.141$ KN/m & 56.015 KN/m (at node i&j)
$P = 45.32$ KN, $P = 30.30$ KN (point load)
$M_i = 2070.3383$ KNm, $M_j = 1570.6216$ KNm
$R_i = 1325.3583$ KN , $R_j = 839.1617$ KN

No. : 8
$l = 6.178$ m, $\omega_{self+wall} = 24.625$ KN/m
$M_i = -M_j = 78.323$ KNm, $R_i = -R_j = 76.067$ KN

No. : 9

$l_x = 10.0$ m, $l_y = 2.0$ m $w_{self} = 12.81$ KN/m (u.d.l.)
$\omega_{slab} = 31.415$ KN/m & 5.235 KN/m (at node i&j)
$P = 15.51$ KN (Point load)
$M_i = 268.2472$ KNm, $M_j = 202.1528$ KNm
$R_i = 245.4643$ KN , $R_j = 145.0957$ KN
$R_{ai} = 55.6367$ KN , $R_{aj} = 32.5935$ KN

3.6 DYNAMIC ANALYSIS :: Force sets ::

Wind forces consideration :

For dynamic analysis wind and earthquake forces are considered. Here the wnd forces comes out to be very small. Even if the section is treated as rectangular the total area comes out to be nearly 175 m^2. And for a force of 104 kg/m^2 total force comes out to be 182 kN.

Earthquake forces consideration :

Place of construction - GORAKHPUR :: Earthquake zone IV

The seismic co-efficint method has been used to calculate dynamic forces for zone IV. During analysis the forces have been used as a lumped mass system.

$$\alpha_h = \beta I \alpha_o$$

$$\beta = 1.0,\ I = 1.5,\ \alpha_o = 0.05 \rightarrow \alpha_h = 0.075 = 7.5\ \%$$

As the dynamic force due to wind is very less in comparison to that due to earthquake only seismic analysis has been carried out here.

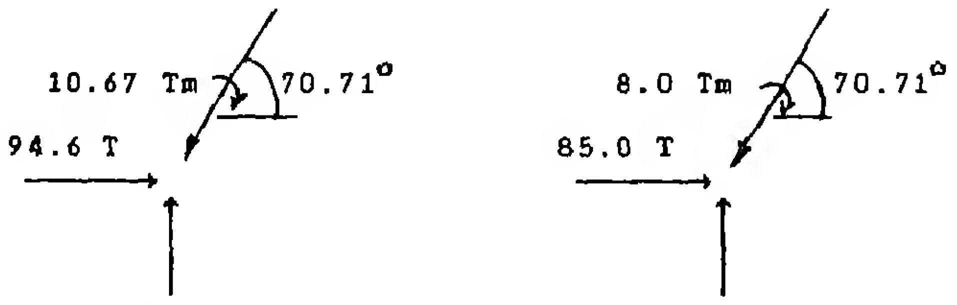
3.7 FOUNDATION DESIGN

PILE SPECIFICATIONS

Diameter : 500 mm. , Spacing : 2*Diameter

3.7.1 DESIGN OF PILE CAP BELOW COLUMNS

Loadings :



Shaft condition: FREE JOINT FIXED JOINT

Load from column : 276 T
Self weight : 30 T
Total weight : 306 T

Is it pile capacity?

?

No. of piles required = 306/60 = 5.1 ≃ 6

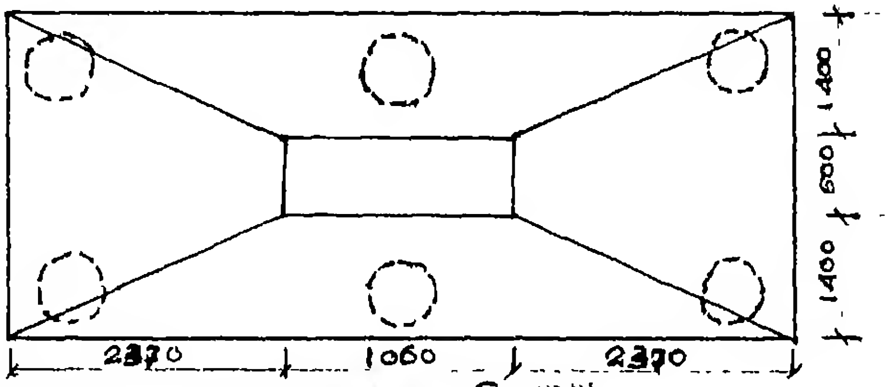


Fig.5 - PILE CAP BELOW COLUMN

Circular punching shear

Let depth of pile cap, d = 25 cm.

Diameter, D = (0.5+0.15+0.125) = 0.75 m = 75 cm.

Perimeter = π*0.75 = 2.356 m = 235.6 cm.

Shear area = 25*235.6 = 5890 cm^2

Shear stress = 51000/5890 = 8.66 kg/cm^2

> 7.16 kg/cm^2

Let depth of pile cap, d = 30 cm.

Diameter, D = (0.5+0.15+0.15) = 0.8 m = 80 cm.

Perimeter = π*0.8 = 2.51 m = 251 cm.

Shear area = 30*251 = 7530 cm^2

Shear stress = 51000/7530 = 6.77 kg/cm^2

< 7.16 kg/cm^2

So provide depth, d = 30 cm.

Total depth = 30+30 = 60 cm.

PUNCHING SHEAR UNDER COLUMN

Punching shear at (at d/2) :

Let depth ,d = 70 cm

Perimeter = (106+35+35)*2+(50+35+35)*2 = 592 cm

Shear area = 592*70 = 41440 cm^2

Shear stress = 306000/41440 = 7.38 kg/cm^2

Let depth ,d = 80 cm

Perimeter = (106+40+40)*2+(50+40+40)*2 = 632 cm

Shear area = 632*80 = 50560 cm^2

Shear stress = 306000/50560 = 6.052 kg/cm^2

< 7.16 kg/cm^2

PUNCHING SHEAR FOR INDIVIDUAL PILE

Punching shear (at d/2) :

Punching shear stress allowed = $0.16\sqrt{f_{ck}}$

= 7.16 Kg/cm^2 for M_{20}

Punching shear force per pile = 306/6 T

= 51 T

Let depth of pile cap, d = 50 cm.

Perimeter = (25+40+25)*4 = 360 cm.

Shear area = 360*50 = 18000 cm^2

Shear stress, = 51000/18000 = 2.833 Kg/cm^2

< 7.16 Kg/cm^2

Let depth of pile cap, d = 35 cm.

Perimeter = (25+40+17.5)*4 = 330 cm

Shear area = 330*35 = 11550 cm^2

Shear stress, = 51000/11550 = 4.416 Kg/cm^2

Let depth of pile cap, d = 30 cm.

Perimeter = (25+40+15)*4 = 320 cm

Shear area = 320*30 = 9600 cm^2

Shear stress, = 51000/9600 = 5.31 Kg/cm^2

< 7.16 Kg/cm^2

Let depth of pile cap, d = 25 cm.

Perimeter = (25+40+12.5)*4 = 310 cm

Shear area = 310*30 = 7750 cm^2

Shear stress, = 51000/7750 = 6.58 Kg/cm^2

< 7.16 Kg/cm^2

Let depth of pile cap, d = 20 cm.

Perimeter = (25+40+10)*4 = 300 cm

Shear area = 320*20 = 6000 cm^2

Shear stress, = 51000/6000 = 8.50 Kg/cm^2

> 7.16 Kg/cm^2

Shear at d

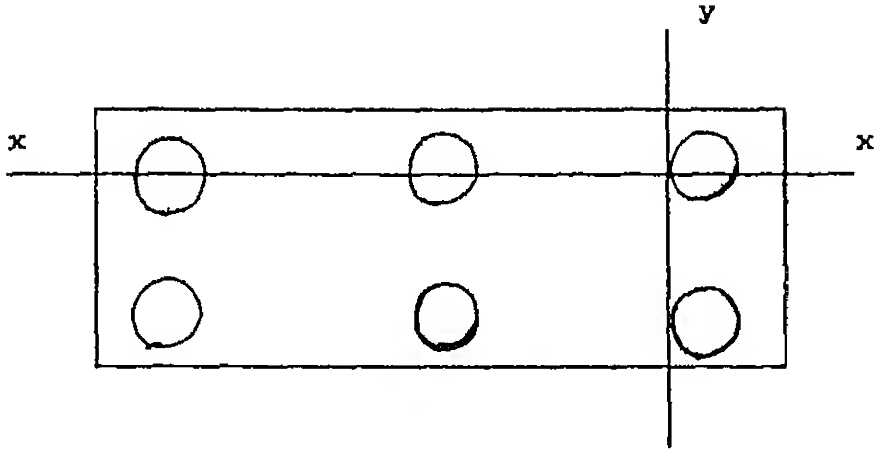


Fig-6 SECTION OF PILE CAP

Along Y axis

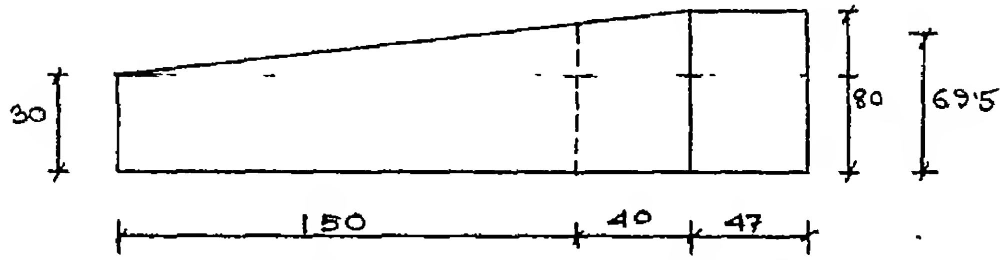


FIG.7A- Elevation of pile-cap. along xx axis

Shear area = 69.5*330 = 22935 cm^2

Shear force = 2*306/6 = 102 T

Shear stress = 102000/22935 = 4.45 kg/cm^2

< 7.16 kg/cm^2

Along X axis

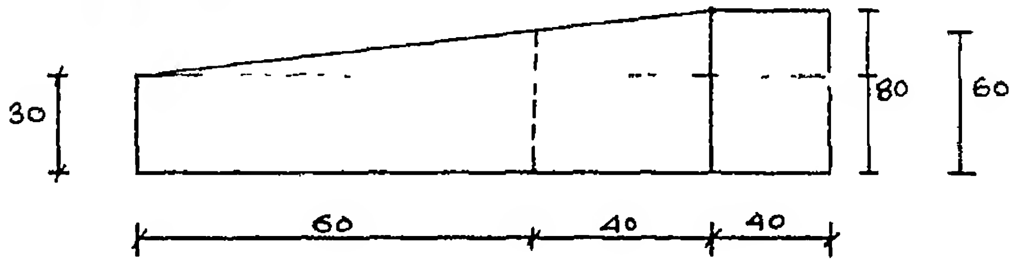


Fig-7B- Elevation along-YY AXIS

Shear area = 60*580 = 34800 cm2

Shear force = 3*306/6 = 153 T

Shear stress = 153000/34800 = 4.40 kg/cm^2

$< 7.16\ kg/cm^2$

DESIGN OF REINFORCEMENT

Along Y axis :

Moment at face = 102*2.37

(downward force) = 241.74 T-m

Moment due to self weight $= 2.5*3.3*(\frac{2.37^2*1.1}{2} - 0.3*1.5*1.87)$

= 25.5 - 6.9 T-m

= 18.5 T-m

Net moment = 223.2 T-m

$$\frac{Mu}{bd^2} = \frac{223.2*1.5*10^7}{3300*800^2} = 1.585$$

==> p_t = 0.494

So, A_{st} = 13041 mm^2 provide 25 ϕ 125 c/c

==> A_{st} = 491*28 = 13748 mm^2 i.e. 0.510 %

τ_c = 0.483 N/mm^2

Shear force = 102-2.5*3.3*(2.37*1.1-0.3-1.5)

= 102-17.8 = 84.2 T

$$\text{Shear stress} = \frac{84.2*1.5*10^4}{3300*800} = 0.478\ N/mm^2$$

Along X axis :

Moment at face = 153*1.4

(downward force) = 213 T-m

Moment due to self weight = $2.5*5.8*(\frac{1.1*1.4^2}{2}-0.3*1.0*1.067)$

= 13.3 T-m

Net moment = 200 T-m

$$\frac{Mu}{bD^2} = \frac{200*1.5*10^7}{5800*750^2} = 0.919$$

==> p_t = 0.282%

So A_{st} = 12180 mm^2 , provide 20 ϕ 125 c/c

==> A_{st} = 314*48 = 15072 mm^2 i.e. 0.361%

τ_c = 0.413 n/mm^2

Shear force = 153-2.5*5.8*(1.0*1.1-0.3*1.0)

= 153-17.98 = 135 T

Shear stress = $\frac{135*1.5*104}{750*5800}$ = 0.465 N/mm^2

Check for reactions

Moments generated :

From static analysis - 10.86 T-m

From dynamic analysis -

SN.	Mx	My	Mz
1	0	0	79
2	1.6	16	69
3	2.8	28	40
4	0	19	62
5	3.2	33	0

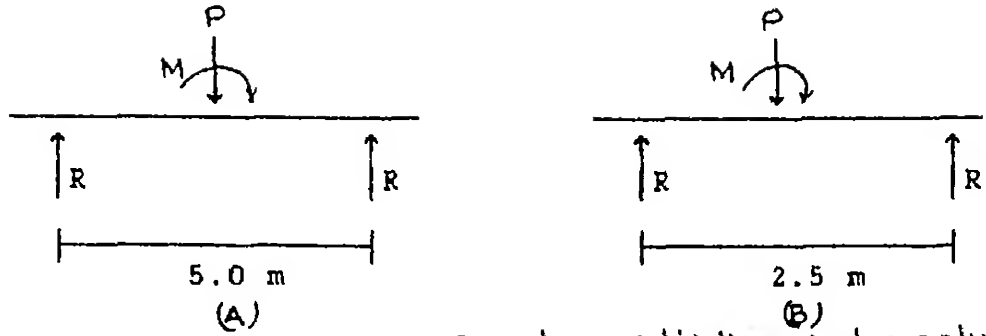


Fig. 8 - Reactions at piles below column

Reactions at the piles :

SN.	R1 (S+D)	R1 (Dyn.)	R2 (Dyn.)	R1+R2 (Max)
1	18.0	0	15.8	18.0
2	15.4	5.9	13.8	21.3
3	10.2	11.2	8.0	21.4
4	14.6	7.6	12.6	22.2
5	0	13.2	0	13.2

(All Values are in Tonnes)

$P_{DL} + P_{LL} = 306/6 + 2.1 = 53.1$ T

$P_{DL} + P_{LL} + P_{eq} = 51 + 22.2 = 73.2$ T

$(P_{DL} + P_{LL} + P_{eq})/1.33 = 54.9$ T $<$ 60 T - (Safe)

3.7.2 DESIGN OF PILE CAP BELOW SHAFT

Loadings :

Shaft connection :	FREE	FIXED
From top beam	12.06 T	15.78 T
From bottom beam	72.27 T	102.8 T
At midlanding	2.08 T	4.49 T
Total	86.41 T	122.07 T
Total for 12 parts	1037 T	1477 T
Self weight	123 T	123 T
Weight of piles	75 T	75 T
Total weight	1237 T	1675 T

Here the weight of the staircases and the domes etc. have not added considering they are small w.r.t. the total weight . For the design purpose the greater value of 1675 T was considered because for the design of pile caps under columns the greater value was taken into consideration. This was done to avoid settlement problem that may be generated if column and shaft have different stiffnesses.

No. of piles required : $1675/60 = 27.9 \simeq 28$

The position of piles are shown in figure :

PUNCHING SHEAR FOR FOUNDATION BELOW SHAFT

Depth at d/2 = 80 cm.

Perimeter = 2* *240 = 1508 cm.

Shear area = 1508*80 = 120640 cm^2

Total force = 840-4*60 = 600 T

Shear stress = 600*103/120640 = 4.97 kg/cm^2

< 7.16 kg/cm^2

So the section is safe against punching shear.

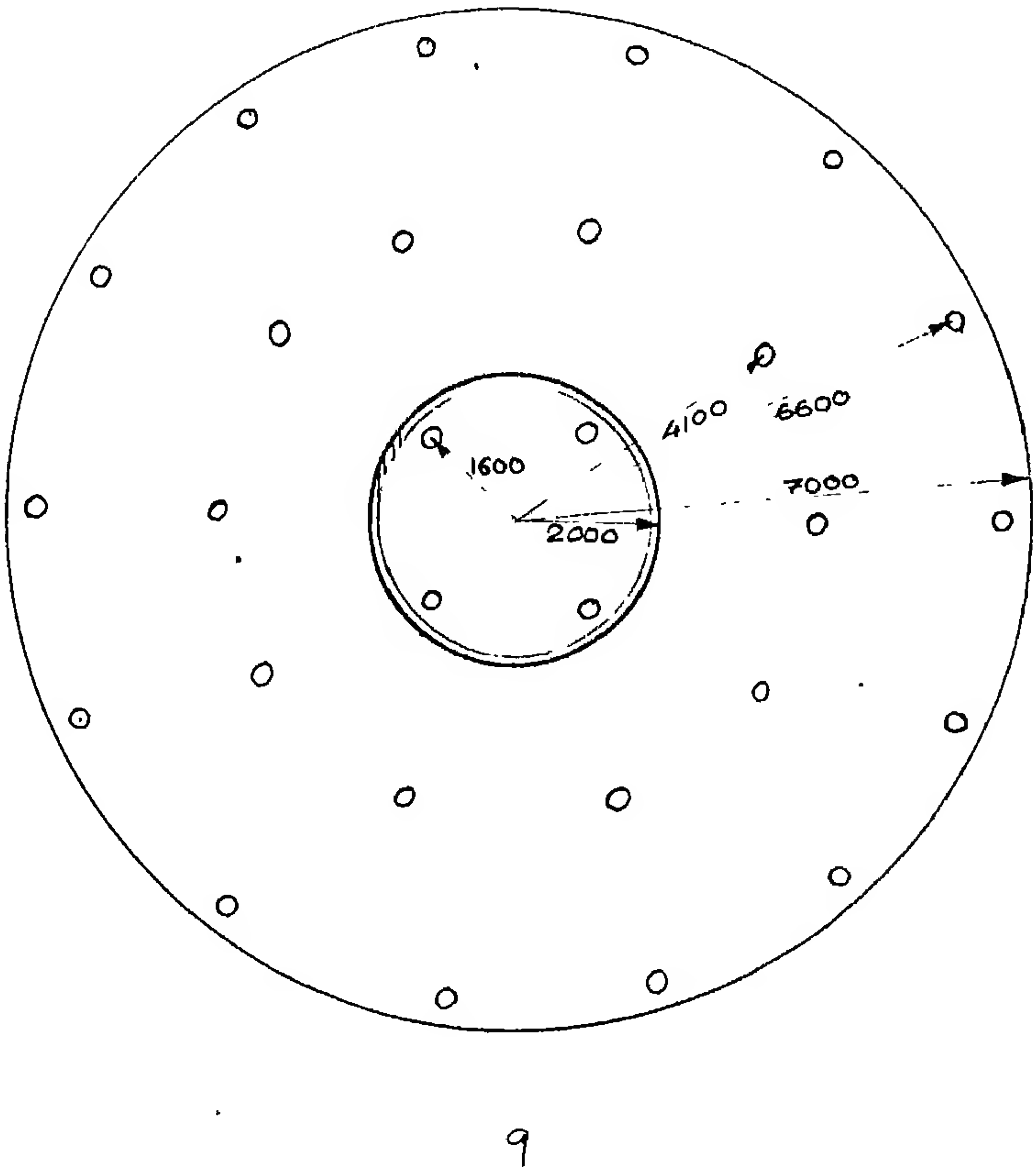
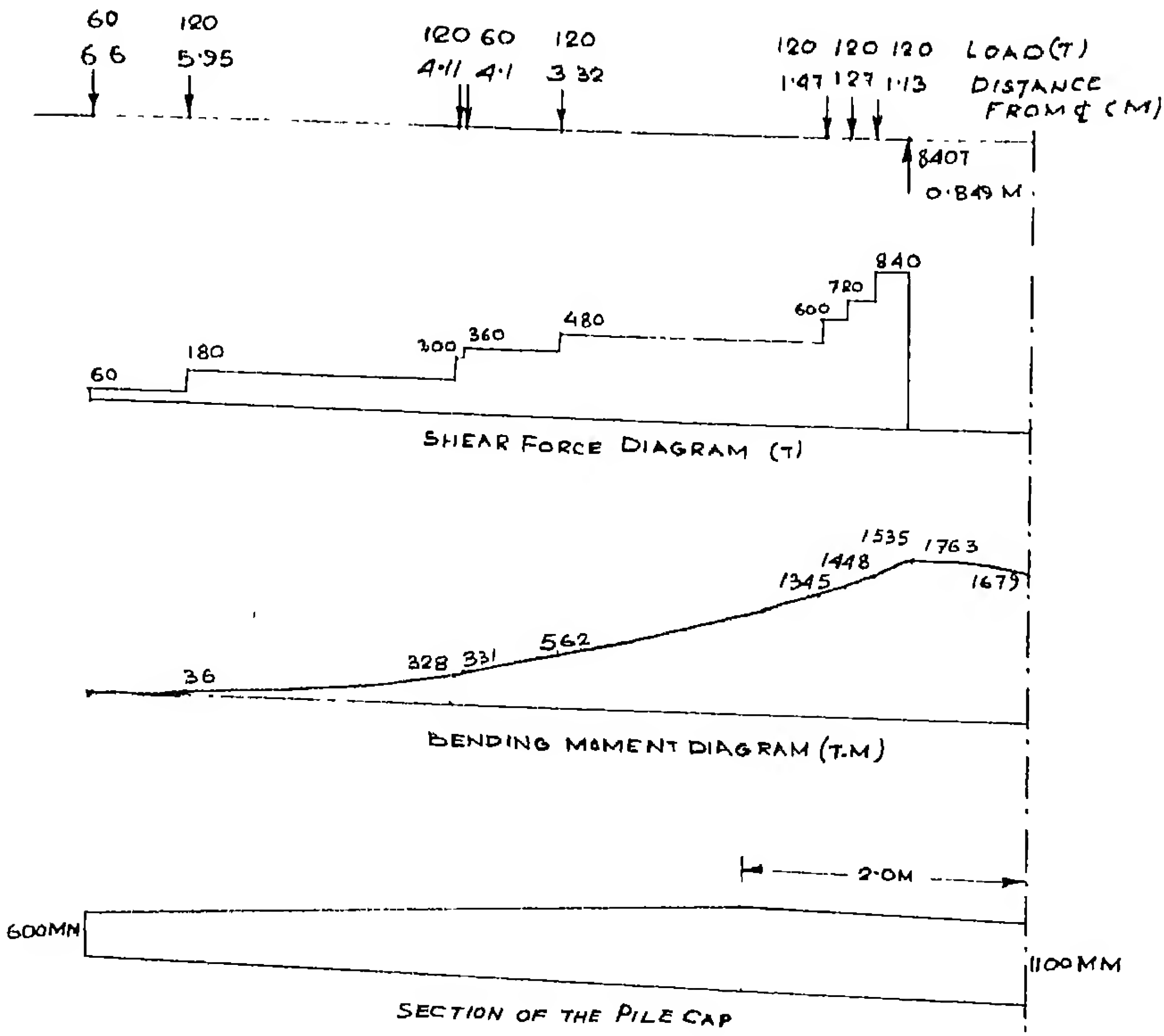


Fig.-9 Position of piles below shaft foundation



Design values

d →	350	510	670	750	850		850		850
$\frac{Mu}{bd^2}$		0 281	096	1·21	2·08	2·18	2 37	2·63	2 48
p_t		0·12	0·30	0·38	0 66	0·72	0·79	0·90	0·83
A_{st}		1020	2507	3196	5567	6095	6715	7924	7063

℄

Fig. 10 - DIAGRAMS SHOWING FORCES ON PILE CAP BELOW SHAFT

3.7.3 DESIGN OF CONNECTING BEAM AT FOUNDATION

Axial force = 96.4 T

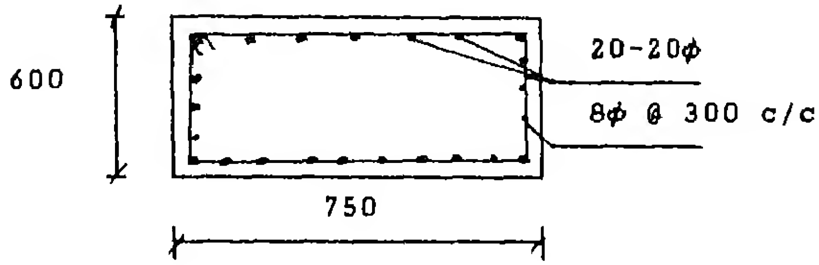


Fig 11 - Section of connecting beam at foundation

Area of steel = 314*20 = 6280 mm2

Total force = 6280*190*10-4+750*600*1.7*10-6

= 120.22 T

≃ 120 T

So the section is safe agaient tension.

3.8 DESIGN OF COLUMNS

Result from static analysis

Col. No.	End 1			End 2		
	P	M_z	V	P	M_z	V
1	291.5	10.67	+0.82	278.4	24.01	-5.41
2	171.5	26.92	+8.47	258.3	42.[illegible]0	-3.74
3	38.61	123.2	+67.27	38.61	35.60	+5.20

Result from dynamic analysis

Col. No.	End 1			End 2		
	P	M_z	V	P	M_z	V
1	40.06	-79.38	-7.11	40.06	60.86	+7.11
2	45.99	89.21	16.63	45.99	109.90	-16.63
3	1.22	-33.56	-10.85	1.22	1.85	10.85

(P: AXIAL FORCE in T, M: MOMENT in T-m, V: SHEAR FORCE in T)

3.8.1 <u>COLUMN NO. 1</u>

Design for static case

End 1 : Axial force = 291.5 T , Moment = 10.67 T-m

End 2 : Axial force = 278.4 T , Moment = 24.01 T-m

$l_x = l_y = \sqrt{(10.5^2+3.675^2)} = 11.125$ m

$l/b = 11.125/0.5 = 22.250 > 12$ --> Slender column

$l/D = 11.125/1.0 = 11.125 < 12$ --> Short column

Minimum eccentricity :

$$e_x = 11125/500+1000/30 = 56 \text{ mm}$$
$$e_y = 11125/500+ 500/30 = 40 \text{ mm}$$

Minimum moments :

End 1 : $M_x = 0.056*291.5 = 16.32$ T-m
$M_y = 0.040*291.5 = 11.66$ T-m

End 2 : $M_x = 0.056*278.4 = 16.09$ T-m
$M_y = 0.040*278.4 = 11.50$ T-m

Additional moments :

End 1 : $$M_{ay} = \frac{291.5*0.5}{2000} (11.125/0.5)^2$$
$$= 36.89 \text{ T-m}$$

End 2 : $$May = \frac{278.4*0.5}{2000} (11.125/0.5)^2$$
$$= 35.24 \text{ T-m}$$

So design parameters for static analysis are ,

End 1 : $P = 291.5$ T, $M_x = 16.32$ T-m, $M_y = 11.66$ T-m
$M_{ay} = 36.89$ T-m

End 2 ; $P = 278.4$ T, $M_x = 24.01$ T-m, $M_y = 11.50$ T-m
$M_{ay} = 35.24$ T-m

Design moment, $M_x = 0.6*24.01+0.4*16.32$ T-m
$= 20.96$ T-m $\simeq 21$ T-m

$$P_{uz} = 0.45*20*500000+0.75*415*14121$$
$$= 450.0+439.519 = 889.52 \text{ T}$$

$$P_u = 1.5*291.5 = 437.5 \text{ T}$$

Determination of P_{bal} :

$d_1/D = 80/1000 = 0.08$ --> $k_1 = 0.207$, $k_2 = 0.328$
$p = 2.8\%$ --> $p_b/f_{ck}bD = 0.253$
$P_{bal} = 252.9$ T --> $K = 0.710$

$$M_{uy} = 0.710*36.89+11.66$$
$$= 37.85 \simeq 38 \text{ T-m}$$

$$P_u/f_{ck}bD = \frac{437*1.5*10^4}{20*500*1000} = 0.432$$

$$M_{ux}/f_{ck}bD^2 = \frac{21*1.5*10^7}{20*500*1000^2} = 0.032$$

$$M_{uy}/f_{ck}Db^2 = \frac{38*1.5*10^7}{20*1000*500^2} = 0.114$$

$\dashrightarrow$ $p/f_{ck} = 0.134$ --> $p = 2.68\%$

Let $p = 2.82\ \%$

$P_u/P_{uz} = 0.492$ --> $\alpha_n = 1.486$

$M_{ux1}/f_{ck}bD^2 = 0.150$ --> $M_{ux1} = 150$ T-m

$M_{uy1}/f_{ck}Db^2 = 0.135$ --> $M_{uy1} = 67.5$ T-m

$$\Longrightarrow (M_{ux}/M_{ux1})^{\alpha_n} + (M_{ux}/M_{ux1})^{\alpha_n} = 0.078$$

So the section is safe at static condition

Results from dynamic analysis —— Load combination ?

...inal in
... Static
...ynamic)
?

End 1 : Axial force = 40.06 T , Moment = 79.38 T-m

End 2 : Axial force = 40.06 T , Moment = 60.86 T-m

Minimum moment $M_y = 0.040*40.06 = 1.60$ T-m

Additional moment: $M_{ay} = \frac{40.06*0.5*}{2000}(11.125/0.5)^2$

$= 4.96$ T-m

So design parameters for dynamic analysis are ,

$P = 331.5$ T, $M_x = 51.00$ T-m, $M_y = 44.41$ T-m

$$P_u = 1.2*331.5 = 397.8 \text{ T}$$
$$M_{ux} = 1.2*51 = 61.2 \text{ T-m}$$
$$M_{uy} = 1.2*44.41 = 53.29 \text{ T-m}$$

$P_u/P_{uz} = 0.447 \quad \text{-->} \quad \alpha_n = 1.412$

$M_{ux1}/f_{ck}bD^2 = 0.150 \quad \text{-->} \quad M_{ux1} = 150$ T-m

$M_{uy1}/f_{ck}Db^2 = 0.135 \quad \text{-->} \quad M_{uy1} = 67.5$ T-m

$$\text{===>} \; (M_{ux}/M_{ux1})^{\alpha_n} + (M_{uy}/M_{uy1})^{\alpha_n} = 0.933$$

So the section is safe at dynamic condition

3.8.2 COLUMN NO. 2

Design for static case

End 1 : Axial force = 271.5 T , Moment = 26.92 T-m
End 2 : Axial force = 258.3 T , Moment = 42.90 T-m

$l_x = l_y = \sqrt{(10.8^2+3.78^2)} = 11.442$ m

$l/b = 11.442/0.5 = 22.884 > 12$ --> Slender column
$l/D = 11.442/1.0 = 11.442 < 12$ --> Short column

Minimum eccentricity :

$e_x = 11442/500+1000/30 = 56.5$ mm
$e_y = 11442/500+ 500/30 = 40$ mm

Minimum moments :

End 1 : $M_x = 0.056*271.5 = 15.34$ T-m
$M_y = 0.040*271.5 = 10.86$ T-m
End 2 : $M_x = 0.056*258.3 = 14.59$ T-m
$M_y = 0.040*258.3 = 10.33$ T-m

Additional moments :

End 1 : $M_{ay} = \frac{271.5*0.5}{2000} *(11.442/5.0)^2$

$= 35.54$ T-m

End 2 : $M_{ay} = \frac{258.3*0.5}{2000} *(11.442/5.0)^2$

$= 33.82$ T-m

So design parameters for static analysis are ,

End 1 : $P = 271.5$ T, $M_x = 26.92$ T-m, $M_y = 10.86$ T-m

$M_{ay} = 35.54$ T-m

End 2 : $P = 258.3$ T, $M_x = 42.90$ T-m, $M_y = 10.33$ T-m

$M_{ay} = 33.82$ T-m

Design moment, $M_x = 0.6*42.90+0.4*26.92$ T-m

$= 36.51$ T-m $\simeq 36.5$ T-m

$P_{uz} = 0.45*20*500000+0.75*415*14121$

$= 450.0+439.519 = 889.52$ T

$P_u = 1.5*271.5 = 407.25$ T

Determination of P_{bal} :

$d_1/D = 80/1000 = 0.08$ --> $k_1 = 0.207$, $k_2 = 0.328$

$p = 2.8\%$ --> $p_b/f_{ck}bD = 0.2529$

$P_{bal} = 264.4$ T --> $K = 0.765$

$M_{uy} = 0.710*35.54+10.86$

$= 36.09 \simeq 36$ T-m

$$P_u/f_{ck}bD = \frac{407*1.5*10^4}{20*500*1000} = 0.41$$

$$M_{ux}/f_{ck}bD^2 = \frac{36.5*1.5*10^7}{20*500*1000^2} = 0.055$$

$$M_{uy}/f_{ck}Db^2 = \frac{36*1.5*10^7}{20*1000*500^2} = 0.108$$

$\rightarrow p/f_{ck} = 0.120 \rightarrow p = 2.40\%$

$p = 2.8\%$

$P_u/P_{uz} = 0.458 \rightarrow \alpha_n = 1.430$

$M_{ux1}/f_{ck}bD^2 = 0.150 \rightarrow M_{ux1} = 150$ T-m

$M_{uy1}/f_{ck}Db^2 = 0.135 \rightarrow M_{uy1} = 67.5$ T-m

$\Longrightarrow (M_{ux}/M_{ux1})^{\alpha_n} + (M_{uy}/M_{uy1})^{\alpha_n} = 0.963$

So the section is safe at static condition

Results from dynamic analysis

End 1 : Axial force = 45.99 T , Moment = 89.26 T-m

End 2 : Axial force = 45.99 T , Moment = 109.9 T-m

Minimum moment $M_y = 0.040*45.99 = 1.84$ T-m

Additional moment: $M_{ay} = \frac{45.99*0.5}{2000}*(11.442/0.50)^2$

$= 6.02$ T-m

So design parameters for dynamic analysis are ,

$P = 317.5$ T, $M_x = 66.50$ T-m, $M_y = 45.80$ T-m

$P_u = 1.2*331.5 = 397.8$ T

$M_{ux} = 1.2*66.25 = 79.50$ T-m

$M_{uy} = 1.2*37.92 = 45.52$ T-m

$P_u/P_{uz} = 0.428 \rightarrow \alpha_n = 1.381$

$M_{ux1}/f_{ck}bD^2 = 0.150 \rightarrow M_{ux1} = 150$ T-m

$M_{uy1}/f_{ck}Db^2 = 0.135 \rightarrow M_{uy1} = 67.5$ T-m

$$\Longrightarrow \left(M_{ux}/M_{ux1}ux \right)^{\alpha_n} + \left(M_{uy}/M_{uy1}uy \right)^{\alpha_n} = 0.996$$

So the section is safe at dynamic condition

3.8.3 COLUMN NO. 3

Design for static case

End 1 : Axial force = 38.61 T , Moment = 123.2 T-m
End 2 : Axial force = 38.61 T , Moment = 35.60 T-m

$l_x = l_y = 5.65$ m

$l/b = 5.65/0.5 = 11.3 < 12$ --> Short column
$l/D = 5.65/1.0 = 5.65 < 12$ --> Short column

END 1:

$$\frac{P_u}{f_{ck}bD} = \frac{38.61*1.5*104}{20*500*1000} = 0.06$$

$$M_{ux}/f_{ck}bD^2 = \frac{123.2*1.5*10^7}{20*500*1000^2} = 0.185$$

$\rightarrow p/f_{ck} = 0.15 \rightarrow p = 3.0\%$

END 2:

$$\frac{Pu}{f_{ck}bD} = \frac{38.61*1.5*10^4}{20*500*600} = 0.06$$

$$\frac{Mux}{f_{ck}bD^2} = \frac{35.60*1.5*10^7}{20*500*600^2} = 0.148$$

(Taking d=600 mm)

$\rightarrow p/f_{ck} = 0.11 \rightarrow p = 2.2\%$

Results from dynamic analysis

End 1 : Axial force = 1.22 T , Moment = 35.56 T-m

End 2 : Axial force = 1.22 T , Moment = 1.85 T-m

Minimum moment M_y = 0.040*40.06 = 1.60 T-m

P_u = 1.2*40.83 = 49 T

The section is safe for dynamic loading because the static loading is much higher than dynamic loading.

3.9 DESIGN OF SHAFT

Total load = 1450 + 10 % = 1595 T

Moment = 0.075*(91.82*24.125+110.3*14.25+20.13*12.0)

= 303 Tm

Let thickness of the shaft = 0.125 m.

Outer diameter = 4.0 m , Inner diameter = 3.75 m

Mean diameter = 3.875 m, Area = 1.5217 m^2

1st moment of area, I = 2.85915 m^4

e = M/w = 0.209 m, e/r = 0.108, τ_{cbc} = 7.0 N/mm^2

$$\Longrightarrow \frac{M}{tr^2 \tau_{cb}} = \frac{3030}{0.125*1.9735^2*7000} = 0.922$$

P_u = 0.4*f_{ck}*A_c + 0.67*f_y*A_{sc}

= 1.21756*107+3.82*106

= 1217+382 = 1599 T

P_{alw} = 1066 T

So the section is not suitable

Let thickness of the shaft = 0.140 m.

Outer diameter = 4.0 m , Inner diameter = 3.72 m

Mean diameter = 3.80 m, Area = 1.6977 m^2

1st moment of area, I = 3.16607 m^4

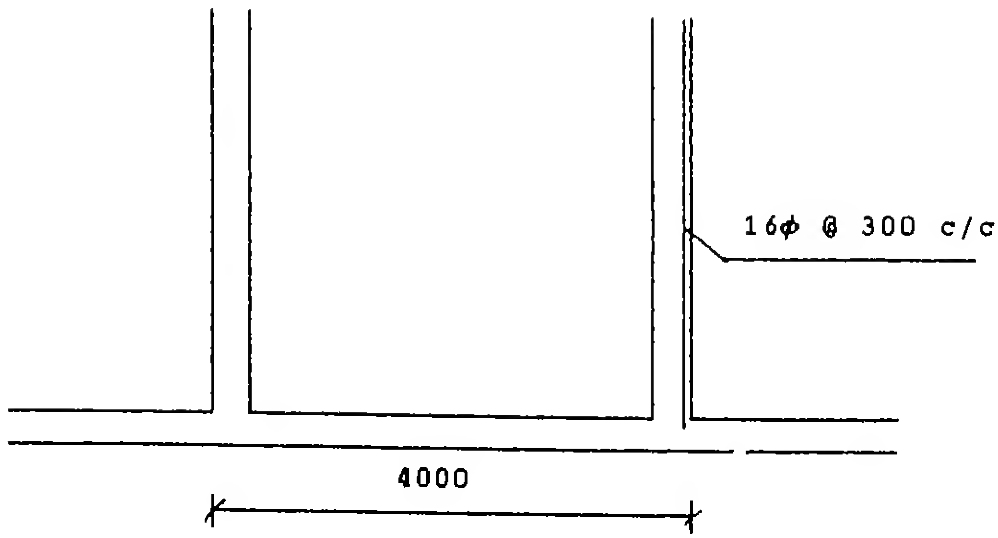
P_u = 0.4*f_{ck}*A_c + 0.67*f_y*A_{sc}

= 1358+1416 = 2774 T

P_a = 1849 T

M = Z*τ_{cbc}

= 1.583*7 = 1108.125 Tm > 303 Tm



Section of shaft

Fig.- 12

3.10 DESIGN OF BEAMS

3.10.1 BOTTOM RADIAL BEAM : RB1

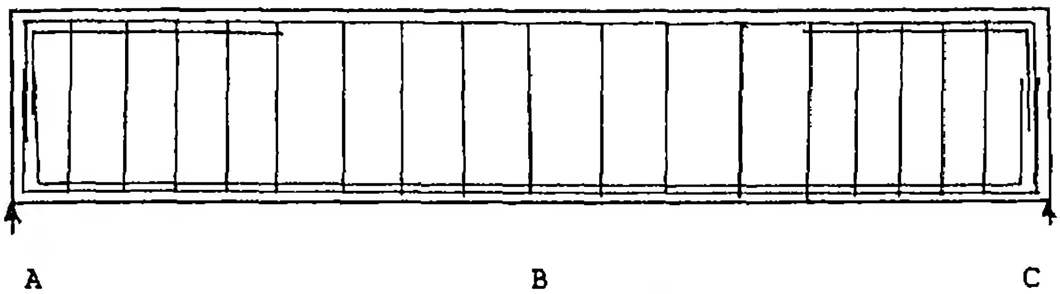


Fig- 13A - SECTION OF A TYPICAL BEAM

	A	B	C
M :	737.4 KNm	1840 KNm	1222 KNm
V :	1288 KN	Negligible	979 KN
T :	776 KN	776 KN	776 KN

Desi gnatend A

M_u = 111 Tm, V_u = 194 T, T_u = 117 T

$A_{st(min)}$ = 0.20% ===> A_{st} = 1500 cm^2 ==> 2-32ϕ

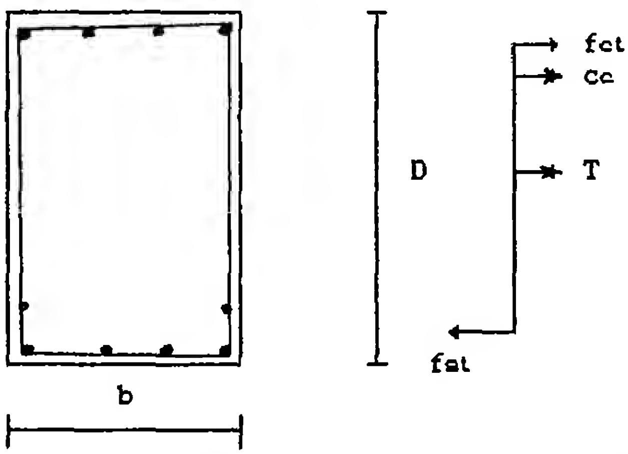


Fig 13B - SECTION OF A TYPICAL BEAM

x_u = 280 mm

==> $c_c = 0.36*280*600*20*10^{-4}$ = 121 T at 118 mm from top

$F_{sc} = 1608*354*10^{-4}$ = 57 T

$F_{st} = 1608*361*10^{-4}$ = 58 T

T_u = 121+57-58 = 120 T
M_u = 121*1.414+57*1.218-120*0.609 = 134 Tm
M_u = 120*0.609+58*1.218-121*0.077 = 134 Tm
So the section is safe in flexture.

A_{st} = 1608 mm^2 ==> $100A_{st}/bd$ = 0.212 ==> τ_c = 0.252
τ_{ve} =$194*10^4/(600*1259)$ = 2.56 N/mm^2 < 2.8 N/mm^2
S_v = 0.87*415/(2.31*600) = 0.26 A_{sv}
So the section is safe in shear

Considering only flexture and shear :

M_u/bd^2 = $111*10^7/(600*1200^2)$ = 1.284
==> p_t = 0.392 ---> A_{st} = 2822 mm^2

Provide 4 - 25ϕ + 4 - 20ϕ (at top)

p_t = 0.43 % , τ_v = 2.613 , S_v = 0.277 A_{sv}

Provide 12 ϕ 4 legged @ 100 c/c.

Designatend C

M_u = 184 Tm, V_u = 147 T, T_u = 117 T
$A_{st(min)}$ = 0.20% ===> A_{st} = 1500 mm^2 ==> 2-32ϕ

Let x_u = 340 mm
==> c_c = $0.36*340*600*20*10^{-4}$ = 147 T at 142 mm from top
F_{sc} = $2594*354.8*10^{-4}$ = 92 T
F_{st} = $3217*361*10^{-4}$ = 116 T

T_u = 147+92-116 = 123 T
M_u = 147*1.116+92*1.218-123*0.609 = 201 Tm
M_u = 123*0.609+116*1.218-147*0.102 = 201 Tm
So the section is safe in flexture.

A_{st} = 3217 mm^2 ==> $100A_{st}/bd$ = 0.425 ==> $_c$ = 0.442
τ_{ve} = $147*10^4/(600*1259)$ = 1.95 N/mm^2 < 2.8 N/mm^2
S_v = 0.87*415/(1.502*600) = 0.40 A_{sv}

Considering only flexture and shear :

$M_u/bd^2 = 184*10^7/(600*1200^7) = 2.13$

==> $p_t = 0.693$ ---> $A_{st} = 4990\ mm^2$

Provide 8 - 25ϕ + 4 - 20ϕ (at top)

$p_t = 0.698\ \%$, $\tau_v = 1.980$, $S_v = 0.418$ Asv

Provide 10 ϕ 4 legged @ 100 c/c.

Designatend B

$M_u = 276$ Tm, $T_u = 117$ T

$A_{st(min)} = 0.20\%$ ===> $A_{st} = 1500\ cm^2$ ==> 2-32 ϕ

$b_f = 7/6 + 0.6 + 0.25*6 = 3.267$ m

Let $x_u = 105$ mm

==> $c_c = 0.36*105*3266*20*10^{-4} = 247$ T at 44 mm from top

$F_{sc} = 1608*324.8*10^{-4} = 52$ T

$F_{st} = (3217+1608)*361*10^{-4} = 174$ T

$T_u = 247+52-174 = 125$ T

$M_u = 247*1.215+52*1.214-125*0.609-58*0.609 = 284$ Tm

$M_u = 58*1.154+116*1.218+0.609*125-247*0.003 = 284$ Tm

So the section is safe in flexture.

Considering only flexture and shear :

$M_u/bd^2 = 276*10^7/(3267*1200^2) = 0.586$

==> $p_t = 0.172$ ---> $A_{st} = 6743\ mm^2$

Provide 12 - 25ϕ + 4 - 20ϕ (at top)

$p_t = 0.180\ \% \rightarrow Sv = 0.317\ A_{sv}$

Provide 12 ϕ 2 legged @ 125 c/c.

3.10.2 TOP RADIAL BEAM : RB2

Place	A	B	c
M :	282 KNm	147 KNm	120 KNm
V :	270 KN	Negligible	120 KN
T :	163 KN	118.5 KN	74 KN

Designatend A

M_u = 42.3 Tm, V_u = 40.5 T, T_u = 24.5 T

Let x_u = 225 mm

==> $c_c = 0.36*225*300*20*10^{-4}$ = 47.5 T at 92 mm from top

$F_{sc} = 402*355*10^{-4}$ = 14.27 T

$F_{st} = 2413*361*10^{-4}$ = 87.11 T

T_u = 87.11-14.27-47.5 = 25.34 T

M_u = 47.5*0.5596+14.27*0.628+25.35*0.302 = 43.2 T-m

M_u = 87.11*0.628-25.35*0.302-47.5*0.0068 = 43.8 T-m

So the section is safe in flexture.

A_{st} = 2413 mm^2 ==> $100A_{st}/bd$ = 1.233 ==> ${}_c$ = 0.62

$\tau_{ve} = 40.5*10^4/(300*652)$ = 2.07 N/mm^2 < 2.8 N/mm^2

S_v = 0.87*415/(1.45*300) = 0.83 A_{sv}

So the section is safe in shear

Provide 4 - 25ϕ + 4 - 20ϕ (at top)

p_t = 0.43 % , τ_v = 2.613 , S_v = 0.277 Asv

Provide 12 ϕ 4 legged @ 100 c/c.

Designatend C

M_u = 18 Tm, V_u = 18 T, T_u = 11.1 T

Let x_u = 75 mm

==> c_c = 0.36*75*300*20*10^{-4} = 16 T at 31.5 mm from top

F_{sc} = 226*342*10^{-4} = 7.73 T-m

F_{st} = 982*361*10^{-4} = 35.45 T-m

T_u = 35.45-16-7.73 = 11.52 T

M_u = 7.73*0.6415+16.2*0.631+11.52*0.302 = 18.66 T-m

M_u = 35.45*0.6415-16.2*0.0105-11.52*0.302= 19.09 T-m

So the section is safe in flexture.

A_{st} = 982 mm^2 ==> 100A_{st}/bd = 0.494 ==> τ_c = 0.475

τ_{ve} = 18*10^4/(300*662) = 0.906 N/mm^2 < 2.8 N/mm^2

S_v = 0.87*415/(0.431*600) = 2.79 A_{sv}

So the section is safe in shear

Provide 8 - 25ϕ + 4 - 20ϕ (at top)

p_t = 0.698 % , τ_v = 1.980 , S_v = 0.418 A_{sv}

Provide 10 ϕ 4 legged @ 100 c/c.

Designatend B

M_u = 276 Tm, T_u = 117 T

$A_{st(min)}$ = 0.20% ===> A_{st} = 1500 cm^2 ==> 2-320/

Let x_u = 115 mm

==> c_c = 0.36*115*300*20*10^{-4} = 24.8 T at 48 mm from top

F_{sc} = 226*342*10^{-4} = 7.73 T

F_{st} = 1432*361*10^{-4} = 51.7 T

T_u = 51.69-7.73-24.8 = 25.61 T

M_u = 7.73*0.631+24.84*0.603+19*0.302 = 25.61 T-m

M_u = 51.69*0.631-24.84*0.0273-19*0.302 = 26.02 T-m

So the section is safe in flexture.

Provide 12 - 25ϕ + 4 - 20ϕ (at top)

p_t = 0.180 % → S_v = 0.317 A_{sv}

Provide 12ϕ 2 legged @ 125 c/c.

3.10.3 BOTTOM CIRCULAR BEAM : CB1

Explain.

The circular beams were idealised as straight beams for analysis purpose.

Axial force = 1282 KN, Shear force = 309 KN, Moment = 177 KNm

As a curved member ,

ω = 9.0+18.375+32.5+46*1.15 = 113.235 KN/m

(for slab, wall, self weight & water)

$M = M_o \cos\theta - \omega r^2 (1-\cos\theta)$

$T = M_o \sin\theta - \omega r^2 (-\sin\theta)$

$M_o = K_4 \omega r^2$, r = 11.6 m, K_4 = 0.04

==> M_o = 609 KNm

--> $\theta = 0^o$: $M = M_o$ = 609 KNm, T = 0

--> $\theta = 15^o$: M = 69.5 KNm, T = 112.33 KNm

So design parameters :

At support - M_f = 177 KNm, M_t = 112 KNm, A =1282 KN, V = 309 KN

At midspan - M_f = 364 KNm, M_t = 0, A (axial force) = 1282 KN

DESIGNATSUPPORT

M_u = 1.5[18+11.2(1+1300/750)/1.7] = 54 Tm

V_u = 1.5[30.9+1.6*11.2/0.75] = 82 T, Tu = 1.5*128.2 = 192.3 T

Let x_u = 455 mm

==> $C_c = 0.36*455*750*20*10^{-4}$ = 245.7 T at 191 mm from top

$F_{sc} = 982*355*10^{-4}$ = 43.86 T

$F_{st} = 2413*361*10^{-4}$ = 87.11 T

==> T_u = 245.7+43.86-87.11 = 193.45 T

M_u = 34.86*1.2215+245.7*1.0679-193.45*0.609 = 187.15 Tm

$100A_{st}/bd$ = 0.255 --> τ_c = 0.362 --> τ_{ve} = 0.868

A_{sv} = $(27*10^7/700+30.9*1.5*10^4)S_v/1221.5*361$
= $(0.875+0.420)S_v$

==> S_v = 0.772 A_{sv} or

S_v = 361/0.506*750 = 0.951 A_{sv}

Design only as a flextural member.

M_u/bd^2 = 0.4608 --> p_t = 0.143
Provide 0.20 % i.e., 1875 mm^2

τ_c = 0.25 %, τ_v = 0.874 N/mm2, S_v = 0.771 A_{sv}
Provide 12ϕ 2 legged @ 150 c/c

DESIGNATMIDSPAN

M_u = 55 Tm, T_u = 193.2 T
b_f = 0.7*.024/12 + 0.75 + 6*0.25 = 2.65 m > 1.8 m

Here the design parameters are same as those of the earlier ones.

3.10.4 DESIGN OF BOTTOM CIRCULAR BEAMS : CB2

l = 4.922 m, w_{self} = 0.4*1.025*2.5 = 1.025 KN/m
w_{slab} = 1.2*0.25*2.5 = 0.75 T/m, w_{water} = 1.2*4.6 = 5.52 T/m
w_{total} = 7.295 KN/m ≃ 7.3 T/m

Fixed end moment = $wl^2/12$ = 14.74 Tm ≃ 15 Tm
Midspan moment = 15/2 = 7.5 Tm
Moment at face = 15-(ωl-ωx)x/2 = 9.93 Tm ≃ 10 Tm

Shear force = ωl/2 = 17.96 T ≃ 18 T
Shear at d/2 = 18-ωd/2 = 15.81 T ≃ 16 T

DESIGNATSUPPORT

M_u/bd^2 = 0.24 for d = 400 mm & 0.32 for d = 300 mm
==> p_t =0.09% --> Provide minimum reinforcement (0.20%)

τ_{va} = 0.64 N/mm^2, τ_c = 0.36 N/mm^2, S_v = 4.30 A_{sv}
--> Provide minimum transverse reinforcement.

DESIGNATMIDSPAN

Here the design parametes are same as those at support, so the design will be same as the design at support. this is also because minimum reinforcements have been provided at the section.

3.10.5 <u>DESIGN OF BOTTOM CIRCULAR BEAMS</u> :CB3:

l = 3.456 m & loading is same as loading of beam CB2

So the disign will be same as beam CB2

3.10.6 <u>DESIGN OF TOP CIRCULAR BEAMS</u> :CB6:

Here also minimum reinforcements should be provided because of small span and smaller laoding.

3.10.7 <u>DESIGN OF BOTTOM CIRCULAR RING</u> :CB4:

This circular ring is to be designed for tension only.
Let force from each beam is T.
Then force in the ring = T(1+2Sin 30+2Sin 60)

So tension = 3.73*76.59 T = 285.83 T

A_{st} = 285.83*1.5*104/0.87*415 = 11875 mm^2
Provide 15 - 32ϕ

3.10.8 DESIGN OF TOP CIRCULAR RING :CB7:

This circular ring also is to be designed for tension only.
Let force from each beam is T.
Then force in the ring = T(1+2Sin 30+2Sin 60) = 3.73 T

So tension = 3.73*16 T = 59.7

A_{st} = 59.7*1.5*104/0.87*415 = 11875 mm^2

Provide 8 - 20ϕ

3.11 DESIGN OF SLAB

Materials : Concrete – M_{20} , Steel – Fe 415

3.11.1 <u>BOTTOM</u> <u>SALB</u> :

The bottom slab is resting on four circuler beams and two radial beams. So it has to be designed in three parts. For the design the annular slab arr idealized as rectanguler slabs where the radial distance and the middle arc length are considered as the two sides of the rectanguler slab. The radial distances between the intermediate circuler beams are adjusted in such a way so as to give nearly equal depth of salb from primary calculations (by traial and error method).

Explain

For preliminary calculations depth of slab is calculated from the equation ,

$$D = \sqrt{(M*6/1700)} \qquad \text{for } M_{20} \text{ concrete}$$

where , D = depth in m.

M = moment in KNm/m

Part A : l_y = 5.498 m ; l_x = 2.2 m ; l_y/l_x = 2.499 > 2.0

So it has to be designed as one way slab.

$M_x = wl^2/12.0 = 53.5*2.2^2/12.0 = 21.699$ KNm/m

So $D = \sqrt{(21.699*6/1700)} = 0.277$ m.

Part B : l_y = 4.489 m ; l_x = 2.8 m ; l_y/l_x = 1.496 < 2.0

So it has to be designed as two way slab.

$-x = 0.053 \implies -M_x = 0.053*2.8^2 = 22.23$ KNm/m

So $D = \sqrt{(22.23*6/1700)} = 0.280$ m.

Part C : l_x = 2.251 m ; l_y = 4.6 m ; l_y/l_x = 2.043 > 2.0

So it has to be designed as one way slab.

$M_x = w*l^2/12.0 = 53.5*2.251^2/12.0 = 22.59$ KNm/m

So $D = \sqrt{(21.699*6/1700)} = 0.283$ m.

So let the design depth is 275 mm

Determination of reinforcements :

According to the INDIAN STANDARD CODE OF PRACTICE the minimum reinforcement is 0.3% for slab of depth 100 mm or less and 0.2% for slab of depth 450 mm or more. For slabs of intermediate depth the percentage of steel should be varied linealy.

Depth of neutral axis,

$$x = (0.5bD^2+(m-1)pbDd)/(bD + (m-1)pbD)$$

$$I_{NA} = bx^2/3 + b(D-x)^2/3 + (m-1)pbD(d-x)^2$$

$$M_R = f_{tb}*I_{NA}/(D-x)$$

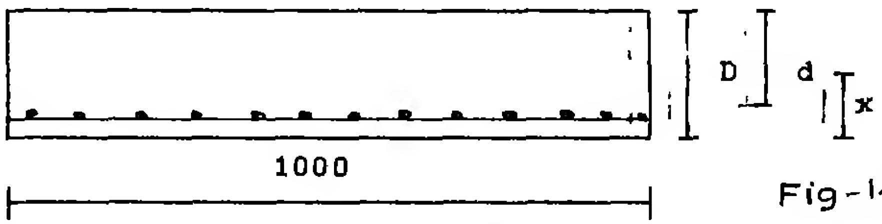


Fig-14A SECTION OF LIQUID RETAINING SLAB

Here let the percentage of steel is 0.3%.

So $p = 0.003$, $D = 275$ mm, $d = 245$ mm

$m= 280/(7*3) = 13$, $f_{tb} = 1.7\ N/mm^2$ for M_{20}

$x = 141.235$ mm, $I_{NA} = 1.844*10^9\ mm^4$

$\Longrightarrow M_R = 23.438$ KNm/m

So reinforcement of 0.3% is provided everywhere.

3.11.2 <u>TOP SLAB</u> :

The bottom slab is resting on three circuler beams and two radial beams . So it has to be designed in two parts.

Part A : $l_y = 6.2$ m ; $l_x = 2.67$ m ; $l_y/l_x = 2.32 > 2.0$

So it has to be designed as one way slab.

$M_x = wl^2/12.0 = 7.0*2.67^2/12.0 = 4.15$ KNm/m

So $D = \sqrt{(4.15*6/1700)} = 0.277$ m.

Part B : l_y = 5.34 m ; l_x = 4.0 m ; l_y/l_x = 1.335 < 2.0

So it has to be designed as two way slab.

- x = 0.049 ==> $-M_x$ = $0.049*4.0^2$ = 5.488 KNm/m

So D = $\sqrt{(5.488*6/1700)}$ = 0.280 m.

So let the design depth is 150 mm.

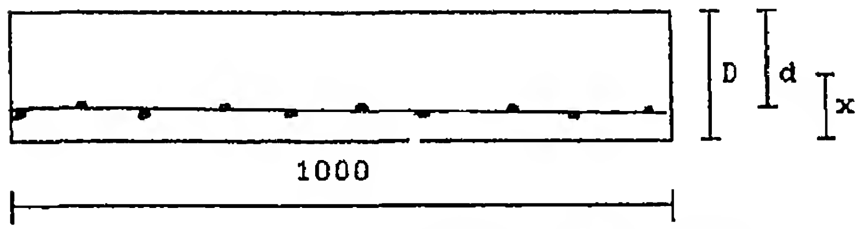


FIG-14b SECTION OF LIQUID RETAING SLAB

Here let the percentage of steel is 0.3%.

So p = 0.003 , D = 275 mm, d = 245 mm

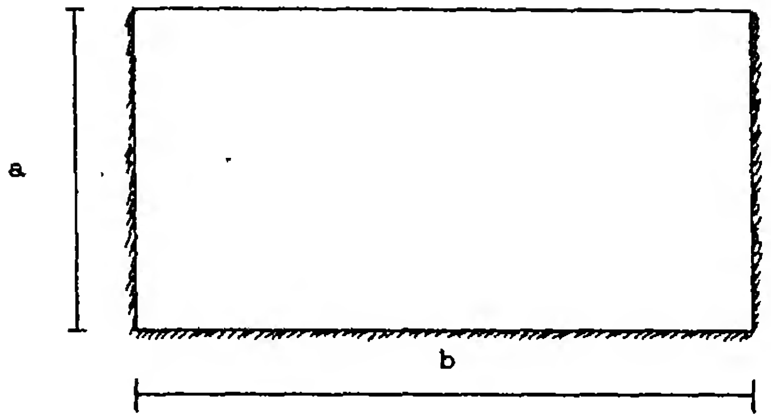
m= 280/(7*3) = 13, f_{tb} = 1.7 N/mm^2

x = 76.7375 mm, I_{NA} = $2.943*10^9$ mm^4

==> M_R = 6.283 KNm/m

So reinforcement of 0.3% is provided everywhere.

3. 12 DESIGN OF TANK WALL



a = 4.6

b = 5.862

ω = 10 kN/m²

b/a = 1.274

Fig 15A - SECTION OF WALL FOR MOMENT COEFFICIENTS

Moment co-efficients for a slab

TABLE-1: MOMENT CO-EFFICIENTS FOR TANK WALL

b/a	x/a	y=0		y=b/4		b/2	
		α_x	α_y	α_x	α_y	α_x	α_y
1.50	0.0	0.0	0.021	0.001	0.005	0.0	-0.040
	0.25	0.008	0.020	0.004	0.007	-0.009	-0.044
	0.05	0.016	0.016	0.010	0.008	-0.008	-0.042
	0.75	-0.003	-0.006	0.003	-0.004	-0.005	-0.026
	1.00	-0.060	-0.012	-0.041	-0.008	0.0	0.0
1.25	0.0	0.0	0.015	0.0	0.003	0.0	-0.029
	0.25	0.005	0.015	0.002	0.005	-0.007	-0.034
	0.5	0.014	0.015	0.008	0.007	-0.007	-0.037
	0.75	0.006	0.007	0.005	0.005	-0.005	-0.024
	1.0	-0.047	-0.009	-0.031	-0.006	0.0	0.0
1.27	0.0	0.0	0.016	0.0	0.003	0.0	-0.030
	0.25	0.005	0.016	0.002	0.005	-0.007	-0.034
	0.5	0.014	0.015	0.008	0.007	-0.007	-0.038
	0.75	0.005	0.006	0.005	0.004	-0.005	-0.024
	1.0	-0.048	-0.009	-0.032	-0.006	0.0	0.0

Moments at the tank wall

$$M_x = \alpha_x \omega a^2 \ , \ M_y = \alpha_y \omega a^2$$

b/a	x/a	y=0		y=b/4		b/2	
		M_x	M_y	M_x	M_y	M_x	M_y
1.27	0.0	0.0	15.18	0.0	3.11	0.0	-29.20
	0.25	5.16	15.08	2.14	5.06	-7.01	-34.07
	0.5	13.82	14.70	7.98	6.91	-6.91	-36.50
	0.75	5.06	5.55	4.67	3.99	-4.87	-23.55
	1.0	-47.01	-9.05	-31.15	-6.03	0.0	0.0

TABLE 2 — MOMENTS IN TANK WALL

Calculation of moment carrying capacity of section

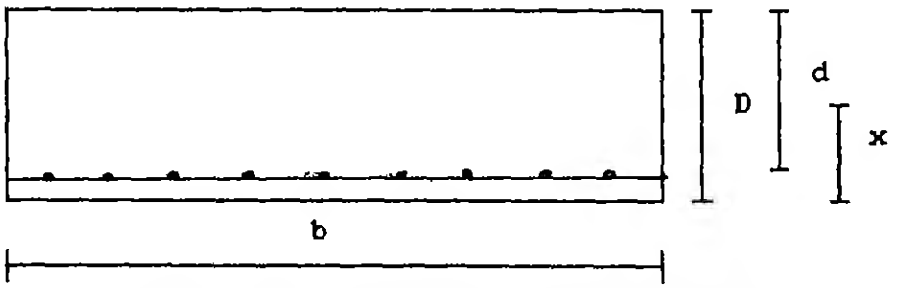


FIG-15B - SECTION OF WALL FOR MOMENTS

DEPTH(D)	DEPTH(d)	STEEL	N.A.(x)	I_{NA}	M_R
(mm)	(mm)	%	(mm)	10^8 mm^4	KNm/m
375	344	0.6	198.0	50.11	48.14
300	269	0.5	156.7	24.90	29.55
318	288	0.6	168.0	30.50	34.41

TABLE.3 — MOMENT RESISTANCE CAPACITY OF SLAB

3.13 DESIGN OF MIDLANDING PLATE :PLT:

This plate has to be designed as compression member.
The force at a section is 3.73 T where T is the force tranferred from the node at column junction.

$T = 128 + 2*78.33*\sin 75$

So compressive force at plate section is,

$= 1042.4*104/250*17000 = 2.45\ N/mm^2$ (safe)

So provide 8 ϕ @ 200 c/c.

3.14 DESIGN OF STAIRCASE :STC:

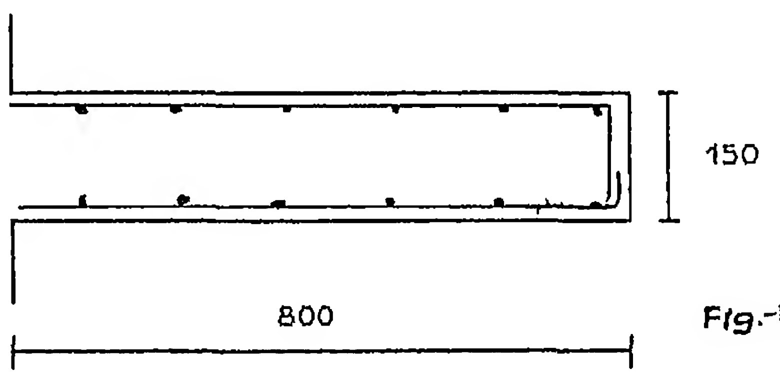


Fig.-16 SECTION OF STAIRCASE

Live load = 300 kg/m^2 = 0.3 T/m^2
Dead load = 2.5*0.15 = 0.375 T/m^2
Railing = 25 kg/m = 0.025 T/m

Total moment = 0.065 Tm ==> M_u/bd^2 = 0.866
p_t = 0.264 %, A_{st} = 39.6 mm^2
Provide 2 - 12 ϕ

3.15 REVISED WEIGHT OF THE WATER TANK

MATERIAL : 1 ::CL1::

Length = √(10.52+3.6752) = 11.125 m
Total length = 11.125*12 = 133.50 m
Weight = 0.5*1.0*133.5*25 = 1668.75 KN
Cummulative weight = 1668.75 KN

MATERIAL : 2 ::CL2::

Length = √(10.82+3.782) = 11.442 m
Total length = 11.442*12 = 137.31 m
Weight = 0.5*1.0*137.3*25 = 1716.36 KN
Cummulative weight = 3385.11 KN

MATERIAL : 3 ::CL3::

Length= 5.65 m
Total length = 5.65*12 = 67.8 m
Weight = 0.5*(1.0+0.7)*67.8*25/2 = 720.34 KN
Cummulative weight = 4105.98 KN

MATERIAL : 4 ::PLT::

Area = π(8.52-22)-1-12*0.5*0.5 = 204.41 m^2
Weight = 201.41*0.3*25 = 1578.10 KN
Cummulative weight = 5684.09 KN

MATERIAL : 5 ::CB1::

Length = 2*π*12.0 = 75.398 m
Weight = 0.750*1.025*25*75.398 = 1449.06 KN
Cummulative weight = 7133.15 KN

MATERIAL : 6 ::RB1::

Length = 12.375-2.0-0.75-0.3 = 9.325 m
Total length = 12*9.375 = 111.9 m
Weight = 111.9*0.6*1.025*25 = 1720.46 KN
Cummulative weight = 8853.61 KN

MATERIAL : 7 ::CB5::

Length = 2*π*12.05 = 75.71 m
Weight = 75.71*0.35*0.5*25 = 331.24 KN
Cummulative weight = 9184.85 KN

MATERIAL : 8 ::RB2::

Length = √(9.72+22) = 9.90 m
Total length = 9.90*12 = 118.85 m
Weight = 118.85*0.3*0.55*25 = 490.25 KN
Cummulative weight = 9675.10 KN

MATERIAL : 9 ::CB2::

Length = 2* *9 4-12*0.5 = 53.06 m
Weight = 53.06*0.3*1.025*25 = 454.04 KN
Cummulative weight = 10129.14 KN

MATERIAL : 10 ::CB3::

Length = 2* *6.6-12*0.5 = 35.47 m
Weight = 35.47*0.3*1.025*25 = 272.67 KN
Cummulative weight = 10401.81 KN

MATERIAL : 11 ::CB4:'

Length = 2* *2.15 = 13.50 m
Weight = 13.50*0.3*1.025*25 = 103.85 KN
Cummulative weight = 10505.66 KN

MATERIAL : 12 ::CB6::

Length = 2* *8.35-12*0.3 = 48.86 m
Weight = 48.86*0.3*0.35*25 = 128.27 KN
Cummulative weight = 10633.93 KN

MATERIAL : 13 ::CB7::

Length = 2* *2.15 = 13.50 m
Weight = 13.50*0.3*0.35*25 = 35.46 KN
Cummulative weight = 10669.39 KN

MATERIAL : 14 ::TWL::

Length = 2* *12.2-12*0.5 = 70.65 m
Weight = 70.65*4.9*0.3375*25 = 2912.14 KN
Cummulative weight = 13590.52 KN

MATERIAL : 15 ::SL1::

Area = *(12.3752-22) = 468.54 m^2
Weight = 468.54*0.275*25 = 3221.20 KN
Cummulative weight = 16811.73 KN

MATERIAL : 16 ::SL2::

Area = *(13.3752-22) = 549.44 m^2
Weight = 549.44*0.15*25 = 2060.38 KN
Cummulative weight = 18872.11 KN

MATERIAL : 17 ::SFT::

Area = *(22-1.862) = 1.6977 m^2
Weight = 29.5*1.6977*25 = 1252.07 KN
Cummulative weight = 20124.18 KN

ESTIMATION OF REINFORCEMENT

ITEM	TYPE	SHAPE	LENGTH	DISTANCE	SPACING	NO.	DIAMETER	TOTAL LENGTH
1. CB1	TOP LONG.	CIRCULAR	62.83			6	20	376.99
	BOT. LONG.	CIRCULAR	62.83			6	20	376.99
	LAP	STRAIGHT	1.04			6X5X2	20	62.40
	FACE	CIRCULAR	62.83			4	12	251.32
	LAP	STRAIGHT	0.624			4X5	12	12.48
	TRANSVERSE	SQUARE	3.844	5.720	175	12X34	8	1568.35
	TRANSVERSE	SQUARE	3.344	5.720	175	12X34	8	1364.35
2. CB2	TOP LONG.	CIRCULAR	50.27			3	16	150.80
	BOT. LONG.	CIRCULAR	50.27			4	16	201.06
	LAP	STRAIGHT	0.832			7X4	16	23.30
	FACE	CIRCULAR	50.27			4	12	201.06
	LAP	STRAIGHT	0.624			4X4	12	9.98
	TRANVERSE	SQUARE	1.472	4.420	250	19X12	8	335.62
3. CB3	TOP LONG.	CIRCULAR	30.16			3	16	90.48
	BOT. LONG.	CIRCULAR	30.16			4	16	120.64
	LAP	STRAIGHT	0.832			7X2	16	11.65
	FACE	CIRCULAR	30.16			4	12	120.64
	LAP	STRAIGHT	0.624			4X2	12	4.99
	TRANVERSE	SQUARE	1.472	3.955	250	17X12	8	300.29
4. CB4	LONGTUDINAL	CIRCULAR	13.509			32	20	432.28
	TRANVERSE	SQUARE	1.472	0.615	300	3X12	8	52.99
5. CB5	TOP LONG.	CIRCULAR	75.71			3	25	227.14
	LAP	STRAIGHT	1.300			3X5	25	19.50
	BOT. LONG.	CIRCULAR	75.71			2	25	151.24
	LAP	STRAIGHT	1.300			2X5	25	13.00
	BOT. LONG.	CIRCULAR	75.71			2	20	151.24
	LAP	STRAIGHT	1.040			2X5	20	10.40

6. CB6	TOP LONG.	CIRCULAR	52.46			2	25	104.93
	LAP	STRAIGHT	1.300			2X4	25	10.40
	BOT. LONG.	CIRCULAR	52.46			2	25	104.93
	LAP	STRAIGHT	1.300			2X4	25	10.40
	TRANSVERSE	SQUARE	1.544	3.870	175	24X12	8	444.67
7. CB7	LONGITUDINAL	CIRCULAR	13.509			8	20	108.07
	TRANSVERSE	SQUARE	1.344	0.615	300	3X12	8	48.38
8. RB1	TOP LONG.	STRAIGHT	11.525			4X12	25	553.20
	TOP LONG.	STRAIGHT	4.275			4X12	12	205.20
	TOP LONG.	STRAIGHT	4.275			4X12	25	205.20
	TOP LONG.	STRAIGHT	2.860			4X12	20	137.80
	BOT. LONG.	STRAIGHT	11.525			6X12	25	829.80
	BOT. LONG.	STRAIGHT	7.775			6X12	25	559.80
	BOT. LONG.	STRAIGHT	6.375			4X12	20	306.00
	FACE	STRAIGHT	10.375			4X12	12	498.00
	TRANSVERSE	SQUARE	3.616	3.200	100	33X12	12	1431.96
	TRANSVERSE	SQUARE	3.416	3.200	100	33X12	12	1352.76
	TRANSVERSE	SQUARE	3.616	3.225	125	27X12	12	1171.56
	TRANSVERSE	SQUARE	3.416	3.225	125	27X12	12	1106.76
	TRANSVERSE	SQUARE	3.580	3.200	100	33X12	10	1417.68
	TRANSVERSE	SQUARE	3.380	3.200	100	33X12	10	1138.48
9. RB2	TOP LONG.	STRAIGHT	12.275			4X12	20	590.16
	TOP LONG.	STRAIGHT	4.375			3X12	25	157.50
	BOT. LONG.	STRAIGHT	12.295			2X12	20	295.08
	BOT. LONG.	STRAIGHT	8.000			2X12	25	192.00
	TRANSVERSE	SQUARE	1.816	3.275	150	23X12	12	501.22
	TRANSVERSE	SQUARE	1.780	6.530	250	27X12	10	576.72
10.F1	TOP LONG.	STRAIGHT	6.150	3.300	125	28X12	25	2066.40
	TOP LONG.	STRAIGHT	3.650	5.800	125	48X12	20	2102.40
11.F2	LONGITUDINAL	STRAIGHT	4.675			20X12	20	1122.00
	TRANSVERSE	SQUARE	2.844	2.475	300	10X12	8	341.28

12.F3	LONGITUDINAL	STRAIGHT	4.000	4.000	100	41X2	32	328.00
	LONGITUDINAL	STRAIGHT	VARIABLE	14.000	100	71X2	25	1463.21
13.SFT	MAIN	STRAIGHT	31.000	11.690	100	118	16	3658.00
	DISTRIBUTION	CIRCULAR	12.068	20.500	200	138	8	1665.38
	DISTRIBUTION	CIRCULAR	12.150	8.450	150	57	10	692.55
14.CL1	MAIN	STRAIGHT	31.891			16X12	32	6123.07
C12	TRANSVERSE	SQUARE	2.944	26.417	250	107X12	8	3780.10
CL3	TRANSVERSE	SQUARE	2.344	26.417	500	107X12X2	8	6619.39
	TRANSVERSE	SQUARE	1.644	26.417	500	107X12X2	8	4221.79
15.SL1	MAIN : B1	STRAIGHT	2.090	5.810	150	39X12	8	978.12
	MAIN : B2	STRAIGHT	3.060	3.870	150	27X12	8	991.44
	MAIN : B4	STRAIGHT	2.252	0.626	150	5X12	8	151.20
	MAIN : B5	STRAIGHT	3.380	3.300	150	22X12	8	892.32
	MAIN : B6	STRAIGHT	1.849	5.900	150	40X12	8	887.50
	MAIN : A1	STRAIGHT	4.050	5.150	150	36X12	10	1749.60
	MAIN : A2	STRAIGHT	5.970	5.150	150	36X12	10	2579.04
	MAIN : A4	STRAIGHT	6.400	2.250	150	16X12	10	1288.80
	MAIN : A5	STRAIGHT	7.390	2.250	150	16X12	10	1418.88
	MAIN : A7	STRAIGHT	4.489	3.300	150	22X12X2	10	2370.19
	MAIN : A8	STRAIGHT	2.549	5.900	150	22X12X2	10	2447.04
	DIST.: B1	STRAIGHT	5.919	0.990	150	8X12	6	710.28
	DIST.: B2	STRAIGHT	3.872	2.760	150	20X12	6	929.29
	DIST.: B3	STRAIGHT	1.168	1.770	150	12X12	6	168.14
	DIST.: B4	STRAIGHT	3.300	2.880	150	20X12	6	792.00
	DIST.: B5	STRAIGHT	5.900	1.349	150	10X12	6	708.00

16.SL2	MAIN : B1	STRAIGHT	1.208	5.460	120	46X12	10	666.54
	MAIN : B2	STRAIGHT	1.628	4.707	120	40X12	10	781.20
	MAIN : B3	STRAIGHT	2.460	3.034	120	26X12	10	767.52
	MAIN : B4	STRAIGHT	2.090	1.042	120	9X12	10	225.72
	MAIN : B5	STRAIGHT	3.613	1.525	120	13X12	10	563.56
	MAIN : B6	STRAIGHT	2.917	2.900	120	25X12	10	875.24
	MAIN : B7	STRAIGHT	1.598	4.300	120	36X12	10	690.34
	MAIN : A1	STRAIGHT	2.750	5.198	120	44X12	12	1452.00
	MAIN : A2	STRAIGHT	3.070	5.189	120	44X12	12	1620.76
	MAIN : A3	STRAIGHT	4.940	4.029	120	34X12	12	2015.52
	MAIN : A4	STRAIGHT	4.408	4.029	120	34X12	12	1789.26
	MAIN : A5	STRAIGHT	5.050	1.830	120	16X12	12	969.26
	MAIN : A6	STRAIGHT	6.370	1.830	120	16X12	12	1223.04
	MAIN : A7	STRAIGHT	7.796	1.525	120	13X12X2	12	2432.25
	MAIN : A8	STRAIGHT	6.289	2.900	120	25X12X2	12	3773.25
	MAIN : A9	STRAIGHT	3.429	4.300	120	36X12X2	12	2962.40
	DIST.: B1	STRAIGHT	5.460	0.478	150	4X12	8	262.08
	DIST.: B2	STRAIGHT	4.709	1.328	150	9X12	8	508.57
	DIST.: B3	STRAIGHT	3.032	2.160	150	15X12	8	548.12
	DIST.: B4	STRAIGHT	1.042	1.770	150	12X12	8	150.05
	DIST.: B5	STRAIGHT	1.525	3.130	150	21X12	8	384.30
	DIST.: B6	STRAIGHT	2.900	2.417	150	17X12	8	591.60
	DIST.: B7	STRAIGHT	4.300	1.098	150	8X12	8	412.80
17.ULL	MAIN : A	STRAIGHT	1.920	5.930	100	60X12	8	1382.40
	MAIN : B	STRAIGHT	2.670	5.930	120	60X12	16	1922.40
	MAIN : C	STRAIGHT	6.900	5.930	100	60X12X2	12	9936.00
	MAIN : D	STRAIGHT	4.058	4.900	140	36X12	16	1753.06
	MAIN : E	STRAIGHT	8.459	4.900	100	50X12X2	12	10150.80
	DIST. : A	CIRCULAR	77.150	1.470	150	10	10	771.50
	DIST. : B	CIRCULAR	77.150	1.470	150	10	10	771.50
	DIST. : C	CIRCULAR	77.150	4.900	150	34	10	2623.10
	DIST. : D	STRAIGHT	4.900	3.558	150	24X12	10	1411.20
	DIST. : E	STRAIGHT	4.900	5.930	150	40X12	10	2352.00
18.PLT	TOTAL	STRAIGHT	VARIABLE	ALTHROUGH	200	86	8	1150.56

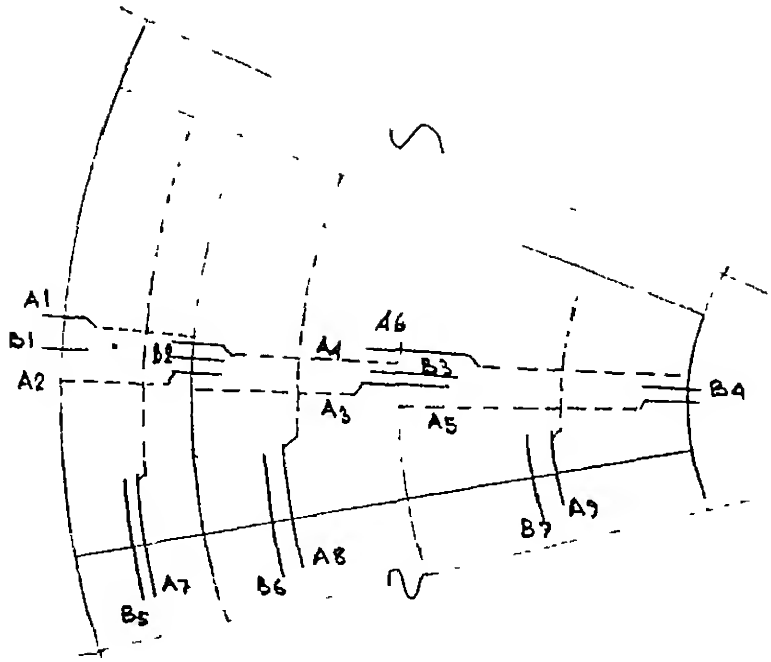


Fig.-17 DIAGRAM FOR DETAILING OF REINFORCEMENT IN SLAB

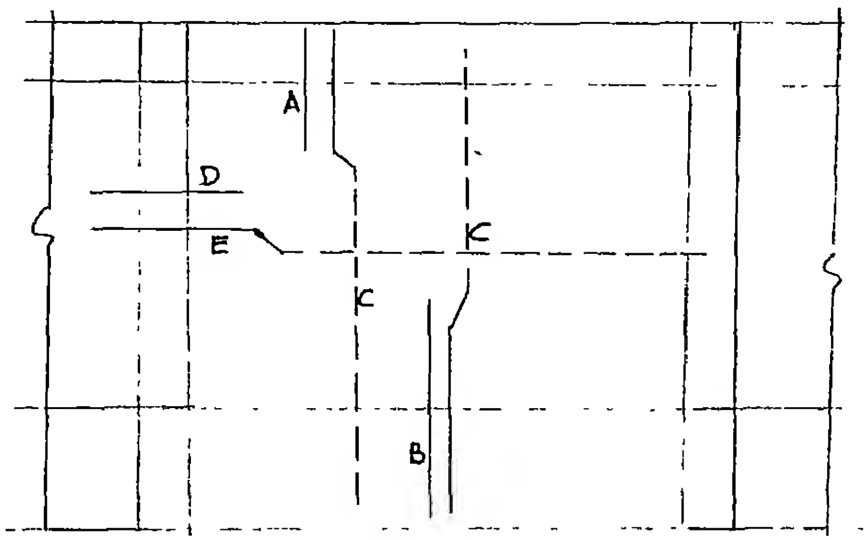


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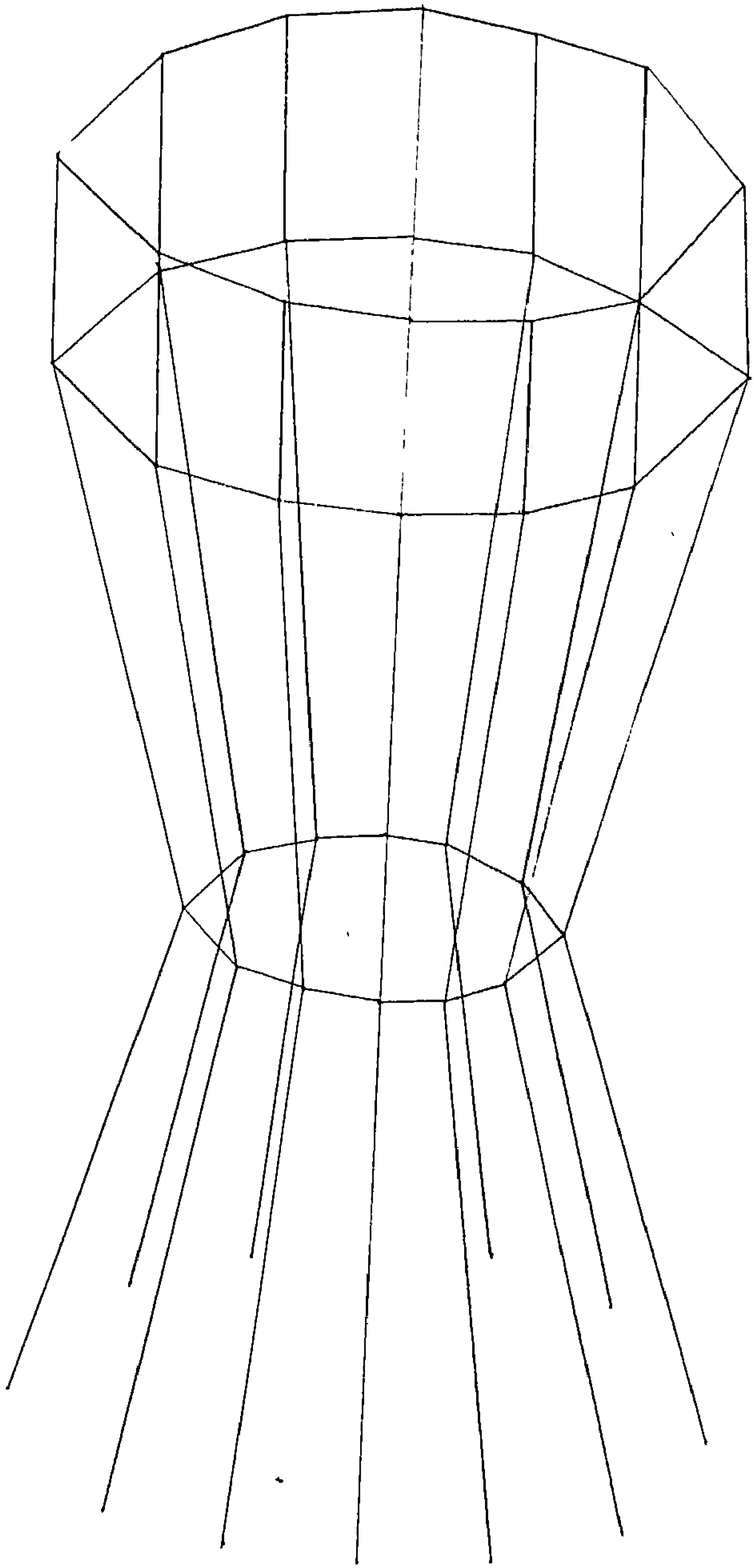
Fig.-18 DIAGRAM FOR DETAILING OF REINFORCEMENT IN WALL

SKETCH FOR [illegible]



SECTION OF TANK FOR ANALYSIS

4.2 STUDY CASES

The parametric study of the water tank analysis is done varying the number of columns, the height of the domes for the lower slab carrying water and the top roof & the diameter at midlanding. The cases are given as below :

Cases according to the radius at midlanding :

1) Radius = 8.5 m.
2) Radius = 9.4 m.
3) Radius = 10.3 m.
4) Radius = 11.2 m.
5) Radius = 12.1 m.

Cases according to the rise of dome :

a) f = 0.0 m. (Flextural member)
b) f = 1.0 m.
c) f = 2.0 m.
d) f = 3.0 m.
e) f = 4.0 m.
f) f = 5.0 m.

Cases according to number of columns :

i) No. of columns = 16
ii) No. of columns = 14
iii) No. of columns = 12
iv) No. of columns = 10
v) No. of columns = 8

4.5. DESIGN PARAMETERS IN COLUMNS

No. of columns = 16

CASE	Flexture P	Flexture Mx1	Flexture Mx2	Flexture P	Flexture Mx1	Flexture Mx2	f=1 P	f=1 Mx1	f=1 Mx2	f=2 P	f=2 Mx1	f=2 Mx2	f=3 P	f=3 Mx1	f=3 Mx2
1	204	2.9	6.1	181	0.4	1.0	182	1.7	7.1	189	2.1	8.8	196	2.8	11.5
2	204	7.2	9.0	181	4.6	3.9	182	11.8	3.0	189	11.2	1.6	196	10.9	7.4
3	205	5.8	13.5	182	3.1	8.6	182	11.6	14.5	189	11.9	13.4	196	11.9	11.8
4	208	6.6	16.6	183	3.9	11.7	183	16.6	23.4	190	16.8	22.5	197	17.1	20.7
5	211	6.9	19.2	185	4.3	14.8	185	19.4	31.0	192	20.1	30.0	200	20.5	28.7

No. of columns = 14

CASE	Flexture P	Flexture Mx1	Flexture Mx2	Flexture P	Flexture Mx1	Flexture Mx2	f=1 P	f=1 Mx1	f=1 Mx2	f=2 P	f=2 Mx1	f=2 Mx2	f=3 P	f=3 Mx1	f=3 Mx2
1	229	3.5	7.1	203	0.8	1.4	200	1.7	7.1	207	2.3	9.2	215	3.2	12.4
2	229	8.0	10.4	203	5.1	4.7	200	11.2	3.1	207	11.1	1.4	215	10.5	1.5
3	231	6.9	15.1	204	3.9	9.6	200	12.0	14.8	207	12.0	13.4	215	11.8	10.8
4	234	7.9	18.3	205	4.8	13.0	201	16.7	23.8	209	17.0	22.6	217	17.1	20.3
5	237	8.4	21.2	207	5.4	16.3	203	19.9	32.0	211	20.4	30.5	220	20.7	28.4

No. of columns = 12

1	263	4.1	8.6	232	1.1	2.3	223	1.9	6.9	231	2.6	9.5	241	3.9	13.4
2	263	9.0	12.1	232	5.8	5.2	223	11.3	3.4	231	10.9	11.6	241	8.6	7.4
3	265	8.3	17.2	233	4.9	11.0	223	12.7	15.4	232	12.1	13.5	241	11.6	10.3
4	268	9.5	20.7	235	6.0	14.7	224	17.2	24.6	233	17.3	22.9	243	17.2	20.0
5	272	10.2	23.8	237	6.8	18.2	227	20.5	32.5	237	20.9	31.0	247	20.9	28.3

No. of columns = 10

1	310	5.0	10.5	273	1.6	3.3	255	1.5	5.9	265	2.6	9.0	277	4.3	13.7
2	310	10.4	14.6	273	6.7	7.3	255	11.8	4.7	265	11.3	2.0	277	9.9	2.3
3	312	10.1	20.1	273	6.2	13.0	255	13.1	17.1	266	12.8	14.7	278	11.9	10.7
4	316	11.7	23.9	275	7.7	16.9	258	18.4	26.5	268	18.3	24.3	280	17.8	20.7
5	321	12.8	27.4	278	8.7	20.8	261	21.9	34.6	272	22.1	32.6	284	21.7	29.2

No. of columns = 8

1	379	6.4	13.3	332	2.3	4.8	303	0.1	1.7	316	1.4	5.4	330	3.5	11.0
2	380	12.5	18.2	332	8.1	9.5	303	13.7	9.2	316	12.7	5.9	330	11.1	0.8
3	382	12.9	24.4	334	8.2	15.8	304	15.6	22.0	317	15.0	19.1	332	13.8	14.7
4	387	14.5	12.9	336	10.1	20.4	307	21.7	31.7	320	20.9	29.1	335	20.0	25.0
5	393	16.5	32.7	340	12.9	23.9	312	25.1	40.7	326	25.0	37.7	341	24.4	33.6

A.4

DESIGN PARAMETERS IN BOTTOM CIRCULAR BEAMS

CASE	M	V	A	M	V	A	M	V	A	M	V	A	M	V	A
No. of columns = 16															
1	5	17.6	34	9	17.6	36	46	40.0	115	56	41.7	166	69	43.7	228
2	6	17.6	44	9	17.6	48	47	40.0	137	57	41.7	189	71	43.7	253
3	6	17.6	55	8	17.6	60	48	40.0	158	59	41.7	212	73	43.7	278
4	7	17.6	64	7	17.6	71	49	40.0	181	59	41.7	237	74	43.7	304
5	8	17.6	74	7	17.6	82	49	40.0	205	60	41.7	263	74	43.7	331
No. of columns = 14															
1	9	20.6	37	7	20.6	40	61	45.7	119	72	47.7	170	88	50.0	232
2	9	20.6	49	7	20.6	53	62	45.7	140	73	47.7	193	89	50.0	257
3	10	20.6	60	6	20.6	65	63	45.7	162	75	47.7	217	91	50.0	282
4	11	20.6	70	6	20.6	78	64	45.7	185	76	47.7	241	92	50.0	308
5	11	20.6	81	5	20.6	90	64	45.7	209	76	47.7	267	93	50.0	335

No. of columns = 12

1	14	24.5	41	5	24.5	44	86	53.3	124	99	55.6	174	117	58.2	236
2	14	24.5	54	4	24.5	58	87	53.3	146	100	55.6	197	119	58.2	261
3	15	24.5	66	4	24.5	72	88	53.3	168	102	55.6	222	120	58.2	287
4	16	24.5	78	3	24.5	86	89	53.3	192	103	55.6	247	121	58.2	313
5	17	24.5	89	2	24.5	96	89	53.3	215	103	55.6	272	122	58.2	340

No. of columns = 10

1	23	30.1	47	1	30.1	49	132	54.0	132	147	66.8	182	169	70.0	243
2	24	30.1	60	2	30.1	65	133	54.0	155	149	66.8	206	171	70.0	269
3	24	30.1	74	3	30.1	80	134	54.0	177	150	66.8	230	172	70.0	296
4	26	30.1	87	3	30.1	96	134	54.0	201	151	66.8	255	173	70.0	321
5	27	30.1	100	4	30.1	111	135	54.0	225	151	66.8	281	174	70.0	349

No. of columns = 8

1	40	38.5	53.4	14	38.5	56	227	80.0	149	247	83.5	196	274	87.4	256
2	41	38.5	69	14	38.5	75	228	80.0	173	248	83.5	221	276	87.4	282
3	42	38.5	85	15	38.5	93	229	80.0	196	250	83.5	246	278	87.4	309
4	43	38.5	99	16	38.5	110	230	80.0	220	251	83.5	271	279	87.4	336
5	45	38.5	114	17	38.5	127	230	80.0	244	251	83.5	297	280	87.4	363

4.5

DESIGN PARAMETERS IN TOP CIRCULAR BEAMS

CASE	M	V	A	M	V	A	M	V	A	M	V	A	M	V	A
No. of columns = 16															
1	4	6.0	33	4	6.0	33	14	17.6	45	14	19.1	65	14	20.7	88
2	4	6.0	34	4	6.0	33	14	17.6	47	14	19.1	67	20	20.7	90
3	4	6.0	36	4	6.0	35	14	17.6	49	14	19.1	69	14	20.7	92
4	4	6.0	38	4	6.0	38	14	17.6	51	14	19.1	71	14	20.7	93
5	4	6.0	41	4	6.0	42	14	17.6	52	14	19.1	72	14	20.7	94
No. of columns = 14															
1	6	6.9	36	6	6.9	35	20	20.1	41	20	21.9	62	20	23.7	86
2	6	6.9	37	6	6.9	36	20	20.1	44	20	21.9	64	20	23.7	88
3	6	6.9	38	6	6.9	38	20	20.1	46	20	21.9	66	20	23.7	89
4	6	6.9	41	6	6.9	41	20	20.1	47	20	21.9	67	20	23.7	90
5	6	6.9	44	6	6.9	45	19	20.1	49	20	21.9	68	20	23.7	91

4·5

No. of columns = 12

1	8	8.0	39	8	8.0	39	30	23.5	37	30	25.5	58	31	27.6	82
2	8	8.0	40	8	8.0	40	29	23.5	38	30	25.5	60	31	27.6	83
3	8	8.0	42	8	8.0	42	29	23.5	40	30	25.5	61	31	27.6	85
4	8	8.0	45	8	8.0	45	29	23.5	42	30	25.5	62	31	27.6	86
5	8	8.0	48	8	8.0	49	29	23.5	43	30	25.5	63	30	27.6	87

No. of columns = 10

1	12	9.6	43	12	9.6	42	47	28.2	27	49	30.5	50	51	33.1	75
2	12	9.6	44	12	9.6	43	47	28.2	29	49	30.5	51	50	33.1	76
3	12	9.6	46	12	9.6	46	47	28.2	30	49	30.5	53	50	33.1	77
4	12	9.6	49	12	9.6	49	47	28.2	32	49	30.5	54	50	33.1	78
5	12	9.6	52	12	9.6	53	47	28.2	33	49	30.5	54	50	33.1	79

No. of columns = 8

1	18	12.1	47	18	12.1	47	85	35.2	8	89	38.2	34	92	41.4	61
2	18	12.1	48	18	12.1	48	85	35.2	10	89	38.2	35	92	41.4	62
3	18	12.1	50	18	12.1	51	85	35.2	11	89	38.2	36	92	41.4	63
4	18	12.1	54	18	12.1	54	85	35.2	12	88	38.2	37	92	41.4	63
5	18	12.1	57	18	12.1	59	85	35.2	13	88	38.2	37	92	41.4	64

The sizes of the members are for the tank supported on less number of columns are assumed to be greater because each member will carry more load if the number of columns are less. This is assumed in case of beam and column sections only, because the thickness of the walls will primarily depend on the depth of water & thickness of bottom slab will depend primarily on the rise of dome and depth of water.

4.6 DESIGN OF TOP ROOF

Rise (f)	1.0	2.0	3.0	4.0	5.0
Radius (r)	12.0	12.0	12.0	12.0	12.0
Radius (R)	72.5	37.0	25.5	20.0	16.9
	9.53	18.92	28.07	36.87	45.24
Sin	0.17	0.32	0.47	0.60	0.71
Depths	200.0	175.0	150.0	125.0	100.0
	175.0	150.0	125.0	100.0	75.0
	150.0	125.0	100.0	75.0	
	125.0	100.0	75.0		
	100.0	75.0			
D.L.	5.0	4.375	3.75	3.125	2.5
	4.375	3.75	3.125	2.5	1.875
	3.75	3.125	2.5	1.875	
	3.125	2.5	1.875		
	2.5	1.875			
L.L.	0.75	0.75	0.75	0.75	0.75
TOTAL	2619	2383	2163	1948	1725
	2358	2092	1863	1634	1394
	2050	1802	1562	1320	
	1765	1511	1262		
	1480	1220			

N	209	97	61	43	32
	189	86	53	36	26
	165	74	44	29	
	141	62	36		
	119	49			
N (min)	240	210	180	150	120
	210	180	150	120	90
	180	150	120	90	
	150	120	90		
	120	90			

So provide for all cases minimum thickness of 75 mm. may be provided. Also minimum reinforcement of 0.3% may be provided in both directions.

4 7 DESIGN OF BOTTOM DOME

Rise (f)	1.0	2.0	3.0	4.0	5.0
Radius (r)	12.0	12.0	12.0	12.0	12.0
Radius (R)	72.5	37.0	25.5	20.0	16.9
	9.53	18.92	28.07	36.87	45.24
Sin	0.17	0.32	0.47	0.60	0.71
Depths	400.0	375.0	350.0	325.0	300.0
	375.0	350.0	325.0	300.0	275.0
	350.0	325.0	300.0	275.0	250.0
	325.0	300.0	275.0	250.0	225.0
	300.0	275.0	250.0	225.0	200.0
D.L.	10.0	9.375	8.75	8.175	7.5
	9.375	8.75	8.175	7.5	6.875
	8.75	8.175	7.5	6.875	6.25
	8.175	7.5	6.875	6.25	5.625
	7.5	6.875	6.25	5.625	5.0
L.L.	20000	20000	20000	20000	20000
TOTAL	24555	24359	24205	24109	23982
	24271	24068	23929	23770	23650
	23986	23801	23605	23456	23318
	23701	23487	23304	23142	22986
	23416	23173	23004	22827	22655

N	1960	996	682	533	448
	1945	984	674	525	441
	1922	973	665	518	436
	1899	960	657	512	429
	1876	948	648	504	423
Ast	25.5	10.0	5.2	3.1	2.0
	27.7	11.2	6.1	3.8	2.8
	29.8	12.5	7.1	4.5	3.8
	32.2	13.9	8.3	5.9	4.9
	34.1	15.6	9.7	7.2	6.4

4.8 DESIGN OF TOP RING BEAM

	1	2	3	4	5
Rise (f)	1.0	2.0	3.0	4.0	5.0
Radius (r)	12.0	12.0	12.0	12.0	12.0
	9.53	18.92	28.07	36.87	45.24
Tension	1188	593	394	293	230
Ast	7920	3953	2627	1953	1533
Steel	10-32	6-25	6-25	4-20	2-20
		+		+	+
Beam	750	700	600	500	400
sizes	X	X	X	X	X
	750	700	600	500	400
pt (%)	1.429	0.857	0.818	0.820	0.895
Beam vol.	43.30	37.71	27.71	19.24	12.31

4.9 DESIGN OF BOTTOM RING BEAM

	1	2	3	4	5
Rise (f)	1.0	2.0	3.0	4.0	5.0
Radius (r)	12.0	12.0	12.0	12.0	12.0
	9.53	18.92	28.07	36.87	45.24
Tension	23195	11306	7721	5116	3785
Ast	154633	75373	51473	34106	25236
Steel (32)	194	92	64	44	34
pt (%)	10.4	5.04	3.43	2.36	1.80
Beam vol.	117.81	117.81	117.81	117.81	117.81

4.10 DESIGN OF FOUNDATION

Rise (mid)	Dia.	COL.-	8	10	12	14	16
4	12.1	W:-	346	289	251	225	204
4	11.2	W:-	346	289	251	225	204
4	10.3	W:-	348	290	252	225	204
4	9.4	W:-	352	293	254	226	205
4	8.5	W:-	358	298	256	229	208
5	12.1	W:-	363	302	263	234	213
5	11.2	W:-	363	302	263	234	213
5	10.3	W:-	366	304	264	234	213
5	9.4	W:-	370	307	266	236	214
5	8.5	W:-	376	313	270	240	218

W : Total load including self weight.

No. of piles	7	6	5	5	4
Sizes (m x m)	5.8X3.3	5.8X3.3	3.3X3.3	3.3X3.3	3.3X3.3
Column(m x m)	1.2X0.65	1.1X0.6	1.0X0.55	0.9X0.5	0.8X0.45
Pile vol.	112	120	120	140	128
Pile cap vol.	132	159	112	112	140
Total vol.	244	279	232	262	268

4.11 DESIGN OF COLUMNS

For the columns the following sizes were taken. It gave reinforcement within specified limit / 22 /. The volume of the columns for a height of 20 m. are also being calculated.

No.	8	10	12	14	16
Sizes(mxm)	1.2X0.65	1.1X0.6	1.0X0.55	0.9X0.5	0.8X0.45
Area	0.78	0.66	0.55	0.45	0.36
Total area	6.24	6.60	6.60	6.30	5.76
Volume	124.8	132	132	126	115.2

4.12 DESIGN OF TANK WALLS

Calculation of level water in the tank

Rise (f)	1.0	2.0	3.0	4.0	5.0
Vol 1	226	457	693	938	1196
Vol 2	226	448	664	871	1065
Vol 3	1774	1552	1336	1129	935
Level 1	3.92	3.43	2.95	2.50	2.07
Level 2	4.92	5.43	5.95	6.50	7.07
Level 3	5.3	5.8	6.3	6.8	7.4

VOL 1 : Volume below the dome curve
VOL 2 : Volume above the dome curve
VOL 3 : Extra volume of water
Level 1 : Water level for extra water
Level 2 : Total depth of water
Level 3 : Total depth of wall

4.12.1 CALCULATION OF MINIMUM THICKNESS OF WALL AT BASE

RISE (f)	1.0	2.0	3.0	4.0	5.0
W.L. (a)	5.0	5.5	6.0	6.5	7.1

Number of comumns : 16 --> b = 4.26 m.

b/a	0.852	0.775	0.710	0.655	0.600
αn	0.0285	0.0251	0.0226	0.0206	0.0186
Moment	35.625	41.76	48.82	56.57	66.57
D max.	355	384	415	447	485
D req.	350	375	400	425	475

Number of columns : 14 --> b = 4.88 m.

b/a	0.976	0.887	0.813	0.751	0.687
αn	0.0339	0.030	0.0268	0.0241	0.0217
Moment	42.375	49.91	57.89	66.18	77.67
D max.	387	420	452	483	524
D req.	375	400	425	475	500

Number of columns : 12 --> b = 5.73 m.

b/a	1.146	1.042	0.955	0.882	0.807
αn	0.042	0.037	0.033	0.0298	0.0265
Moment	52.5	61.56	71.28	82.84	96.85
D max.	430	466	502	541	584
D req.	425	450	475	525	550

Number of columns : 10 --> b = 6.84 m.

b/a	1.368	1.244	1.140	1.052	0.963
αn	0.053	0.0467	0.0417	0.0375	0.0334
Moment	66.25	77.70	90.07	102.98	119.54
D max.	484	524	564	603	650
D req.	475	500	550	575	625

Number of columns : 8 --> b = 8.67 m.

b/a	1.737	1.576	1.445	1.334	1.220
αn	0.0733	0.0643	0.0571	0.0514	0.0456
Moment	91.625	106.98	123.42	141.16	163.21
D max.	569	614	660	706	759
D req.	550	575	625	675	725

TABLE - 4

TABLE -5

4.12.2 CALCULATION OF MINIMUM THICKNESS OF WALL AT MIDLEVEL

RISE (f)	1.0	2.0	3.0	4.0	5.0
W.L. (a)	5.0	5.5	6.0	6.5	7.1

Number of columns : 16 --> b = 4.16 m.

b/a	0.852	0.775	0.710	0.655	0.600
n	0.0217	0.0182	0.0157	0.014	0.0122
Moment	27.375	30.28	33.91	38.45	43.67
D max.	310	327	346	368	393
D req.	300	325	325	350	375

Number of columns : 14 --> b = 4.88 m.

b/a	0.976	0.887	0.813	0.751	0.687
αn	0.0278	0.0235	0.0200	0.0171	0.0150
Moment	34.75	39.10	43.20	46.96	53.69
D max.	350	371	390	407	435
D req.	325	350	375	400	425

Number of columns : 12 --> b = 5.73 m.

b/a	1.146	1.042	0.955	0.882	0.807
αn	0.0337	0.0303	0.0268	0.0233	0.0197
Moment	42.125	50.41	57.89	63.99	70.51
D max.	386	422	452	475	499
D req.	375	400	425	450	475

Number of columns : 10 --> b = 6.84 m.

b/a	1.368	1.244	1.140	1.052	0.963
αn	0.0394	0.0368	0.0335	0.0307	0.0272
Moment	49.25	61.23	72.36	84.31	97.35
D max.	417	465	505	545	586
D req.	400	450	475	525	550

Number of columns : 8 --> b = 8.67 m.

b/a	1.737	1.576	1.445	1.334	1.220
αn	0.0516	0.0464	0.0418	0.0387	0.0360
Moment	64.5	77.29	90.29	106.28	128.85
D max.	477	522	564	612	674
D req.	450	500	550	600	650

Here the moments are in kNm/m and the depths are in mm. The reinforcements are varying from 0.2% (min.) to 0.5%. Calculation of depth and reinforcements taking only tension

TABLE-6

4.12.3 DESIGN OF TANK WALL AS A TENSION MEMBER

RISE (f)	1.0	2.0	3.0	4.0	5.0
W.L. (a)	5.0	5.5	6.0	6.5	7.1
Wn (kN/m/m)	50.0	55.0	60.0	65 0	71.0
T (kN/m)	600.0	660.0	720.0	780.0	852.0
Ast	4000.0	4440.0	4800.0	5200.0	5680.0
Depth	500.0	550.0	600.0	650.0	710.0
p (%)	0.63	0.63	0.63	0.63	0.63

So the tank walls should be designed as two way slab i.e. as flextural member to reduce the depth as well as percentage of steel.

TABLE-7

4.12.4 VOLUME OF TANK WALL FOR DIFFERENT CASES

RISE (f)	1.0	2.0	3.0	4.0	5.0
HEIGHT	5.3	5.8	6.3	6.8	7.4
16 Column	132.903	156.630	178.639	203.310	242.659
14 Column	143.129	167.817	194.437	229.544	264.091
12 Column	163.574	190.193	218.741	255.777	292.620
10 Column	178.155	212.569	249.122	288.569	335.442
8 Column	204.562	240.538	285.579	334.477	392.539

4.13 CALCULATION

4.13.3 Calculation for designed tank

Amount of reinforcement used

Dia. (mm)	Length (m)	Vol. cu m.
6	3307.71	0.0093
8	30033.63	1.5092
10	28171.40	2.2126
12	45193.43	5.1113
16	7831.39	1.5745
20	6072.44	1.9077
25	6668.65	3.2735
32	6451.07	5.1883

TOTAL VOLUME --> 20.8712 m^3

Volume of concrete to be used in the tank

Substructure :
Below column - 195
Below shaft - 109
Tension beam - 12.3
Piles - 60.5
TOTAL - 376.8

Superstructure :
Weight/unit wt. = 20124.18/25
= 804.26

TOTAL :: 1181.76 m^3

4.13.2 Calculation for optimum shape

A rise of less than 4 m. in the bottom dome increases the steel percentage beyond the permissible limit specified / 22 /. So this cases are neglected. Again if the midlanding diameter is decreased circular beam or radial beams have to be provided which will increase the cost. So straight columns are considered for staging.

Calculation of volume :

No. of columns :	8	10	12	14	16
Vol. of column	124.8	132	132	126	115.2
Vol. of wall*	334	288	255	229	203
Vol. of wall**	392	335	292	264	242
Vol.of foundation	244	279	232	262	268
Vol. of beams*	136	136	136	136	136
Vol. of beams**	130	130	130	130	130
Vol. of domes*	187	187	187	187	187
Vol. of domes**	167	167	167	167	167
Total *	1026	1023	943	941	920
Total **	1067	1053	926	959	942

* - For a rise of 4 m.
** - For a rise of 5 m.

Therefore saving that could have been done is around 261 m^3 i.e., 28.6 %.

4.14 STUDY OF RESULTS

4.14.1 DESIGN OF ROOF SLAB

Design parameters of shell structure generally depends on the curvature of the surface and the loadings. In case circular domes the design depends on the radius at the base and the height of rise of the dome. In this case the design depends primarily on the rise of the dome as the radius is fixed. Here due to very small amount of load a minimum depth 75 mm and a minimum reinforcement of 0.30% is sufficient enough for the serviceability of the system.

In comparison, in the flat flextural slab the depth depends on the length of the beams on which it is supported which depends on the number of columns on which the beams are supported. For the present problem a minimum depth of 150 mm is required along with an intermediate circular beam.

4.14.2 DESIGN OF BOTTOM SLAB

It has been shown earlier that design of bottom slab is very related with the rise of dome. For a radius of 12 m the suitable rise is 4 m & 5 m. For less rise there is a steep increase in the requirement of steel. The percentage of steel goes up from a moderate value of 2.04% at a rise of 5.0 m to an unacceptable value of 35.1% at a rise of 1.0 m while the depth is kept at a fixed value of 300 mm. Even if the depth is increased to 400 mm the requirement of steel

comes down only to 28%. So it is always adviseable to have more rise in domes. The recommended value is of the order of one-sixth to one-seventh of the base diameter (1).

4.14.3 DESIGN OF TANK WALL

The depth of tank wall varies with the rise of the dome and the radius of curvature of the surface.

In has been shown that the material requirement is less for the design of the wall as flextural member supported on the beam and columns. So naturally the depth of depends on the beams while other parameters are fixed. And the length of beams depends on the number of columns on which it is supported.

Here ir is adviseable to use less rise of the dome, because the increase in the rise of dome increases the depth of water in th tank. This increases the loaded area in the slab which in turn increases the depth of the slab. For the present case it would be better to take rise of 4 m.

4.14.4 DESIGN OF TOP CIRCULAR BEAM

Analysis by F.E.M. is less accurate here because the curved members are taken as straight members and that too is defined by two points only. Here the circular members

are designed as ring beam under tension. Due to very less load the rise of the dome has a very little effect in the design of the beams.

4.14.5 DESIGN OF BOTTOM CIRCULAR BEAM

The bottom circular beam is also designed as ring beam under tension. Here the percentage of steel requirements are very high in case of low rise of the domes. Axial forces in the beams are considerably less in case of F.E.M. analysis because of idealization of the members.

Here also it is adviseable to use higher rise in the dome to reduce the percentage of steel keeping the beam size fixed.

4.14.6 DESIGN OF COLUMNS

It is seen here that the axial force in the columns depends on the diameter at midlanding, rise of dome, number of columns in the systems. It is recommended here (based on the results) to have vertical columns because the design parameters increases in inclined columns. The gradient is small at the beginning and increases rapidly afterwards (fig.-). It is also recommended to have two more bracings in two more levels because relatively higher additional moment has to be considered in minor axis where the moments in the major axis is nearer to the moment due to

Explain

Explain

minimum eccentricity.

The trend in the curves show that if the diameter in midlanding is increased more than the diameter at base of tank the force will increase because the tension will be generated at the beams.

4.11.7 DESIGN OF FOUNDATION

It is adviseable to use vertical columns for support because otherwise shear force is generated. The shear force increases with the increase of the angle of inclination of the columns. ? Expl-

Design has shown that vertical columns requires least amount of material.

For the present design the increase in cost ~~in~~ is primarily due to the shaft. The construction of shaft and its foundation will require large amount of material. Indirectly it also affects the design of the ~~design of~~ radial beams, because the connection of the radial beams at the shaft can't be established properly. It is neither a fixed joint nor a free joint. If joint considered fixed the value of the design parameters in the radial beams at the shaft increases very much whereas if the joint is considered as free then the value of the design parameters increases in all other members.

The presence of the radial beams increases the dead load

in the system and as a result requirement of steel increases in the columns. Again from the point of view analysis in computers the flat slab system requires more time in computers.

The only question which may be raised here regarding the safety of the system is the joint condition consideration of the radial beams at the shaft. The superstructure is designed with the higher values of the design parameters except the radial beams. The substructure and the radial beams have designed taking intermediate values.

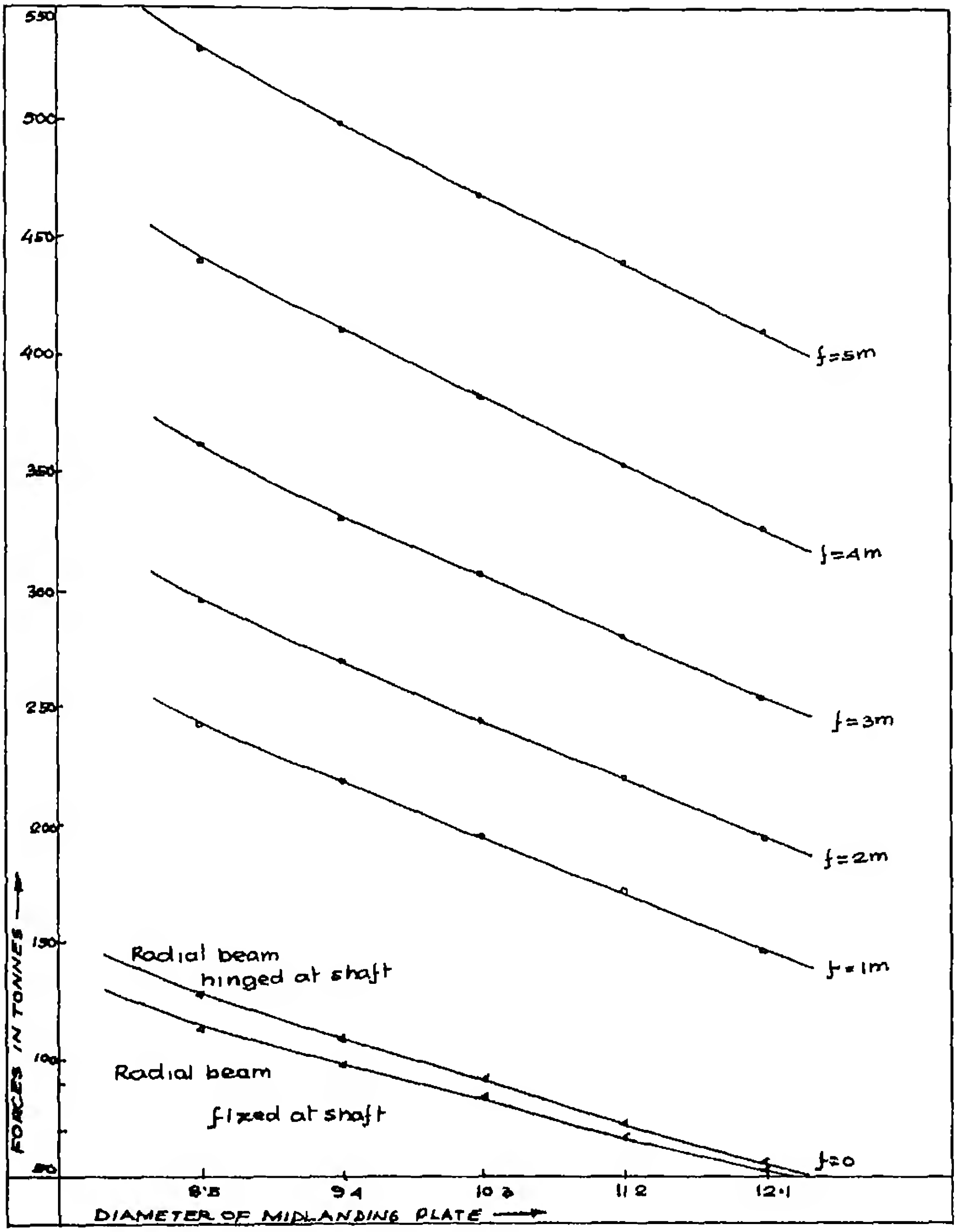


Fig - 19A Tensile force in bottom circular beam
(No. of columns - 8)

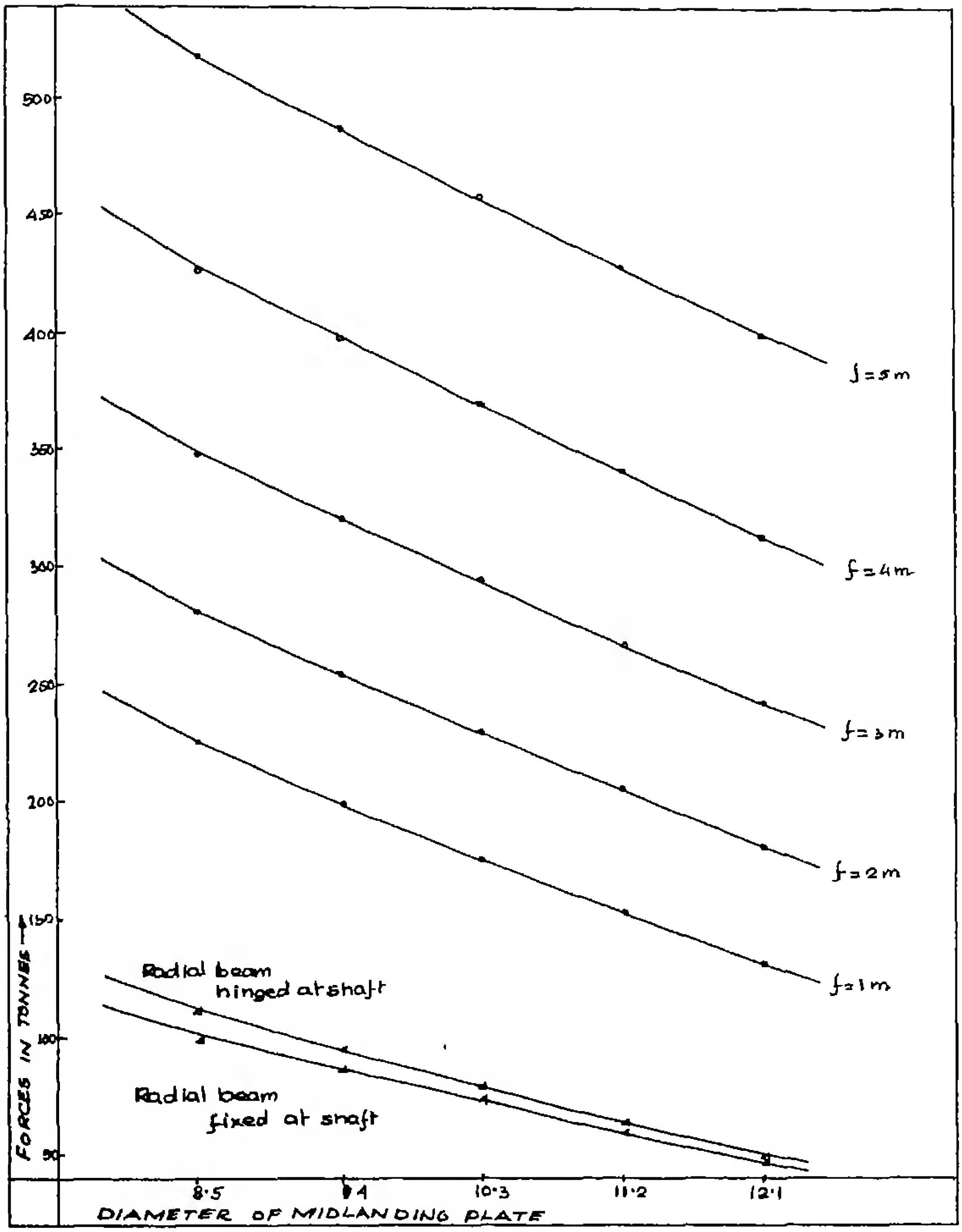


Fig.– 13B TENSILE FORCE IN BOTTOM CIRCULAR BEAM
(NO of columns -10)

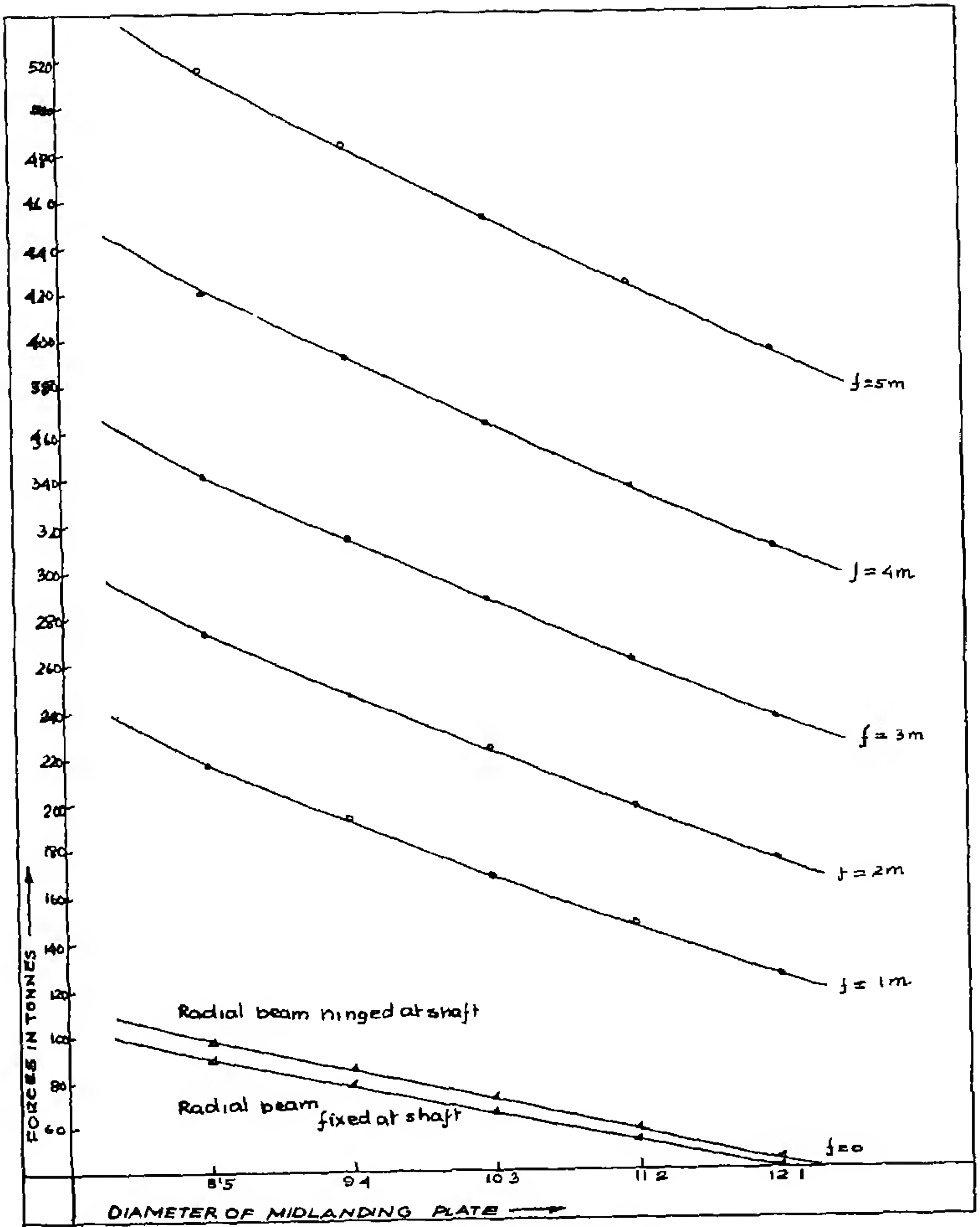


Fig - 19c Tensile force in bottom circular beam
(No of columns - 12)

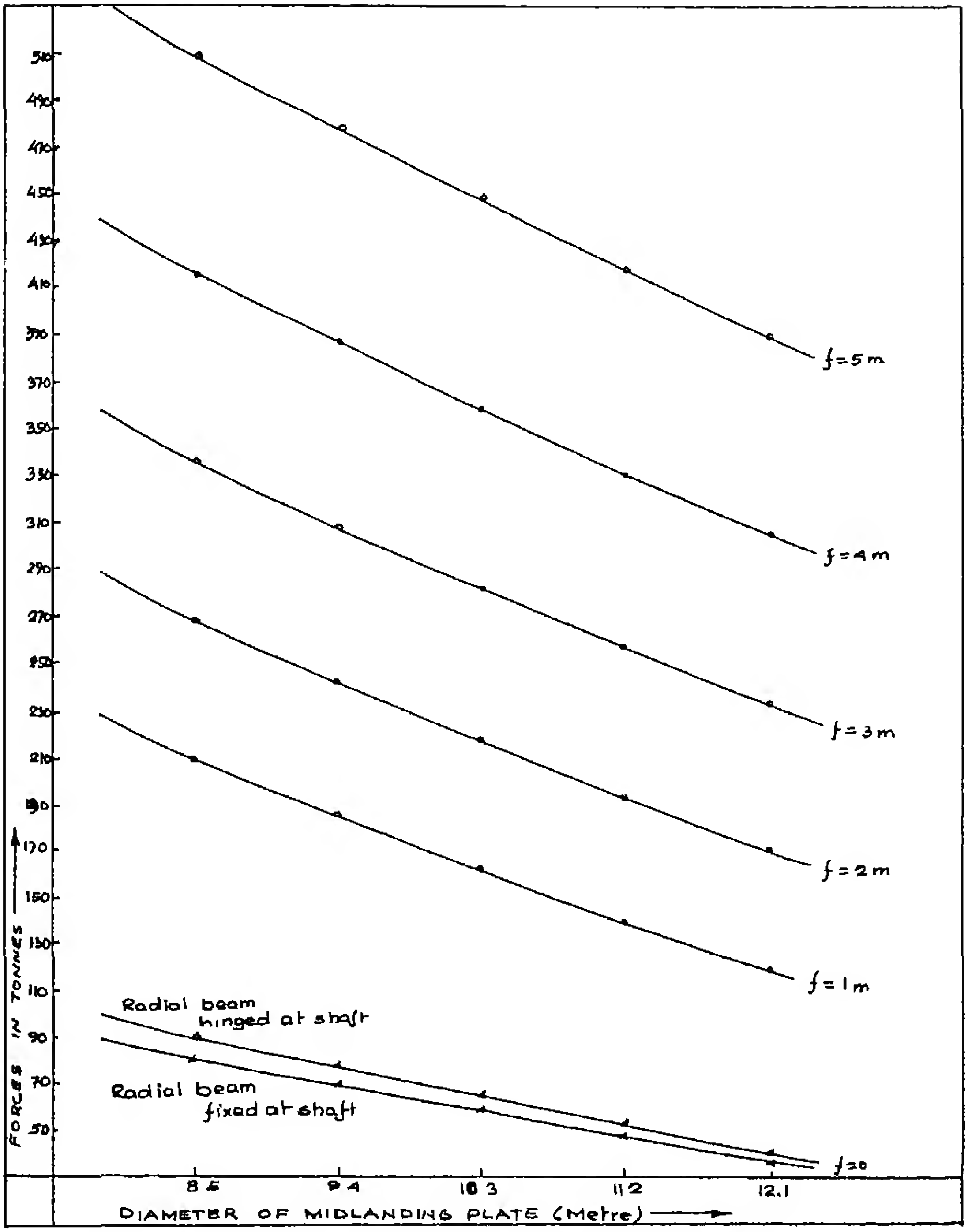


Fig.-19D : TENSILE FORCES IN BOTTOM CIRCULAR BEAM
(NUMBER OF COLUMNS - 14)

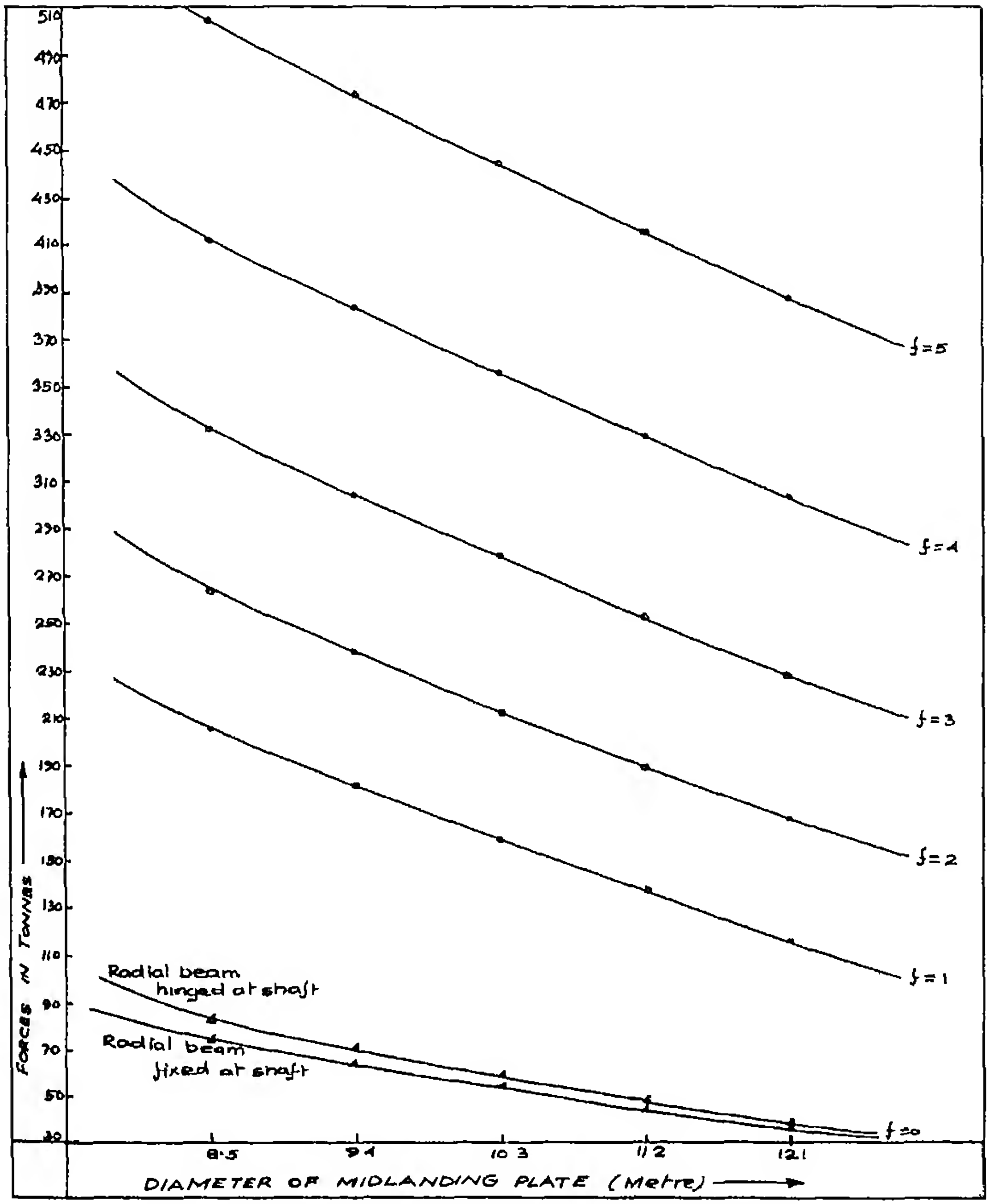


Fig - 19E . TENSILE FORCE IN BOTTOM CIRCULAR BEAM (NUMBER OF COLUMNS - 16)

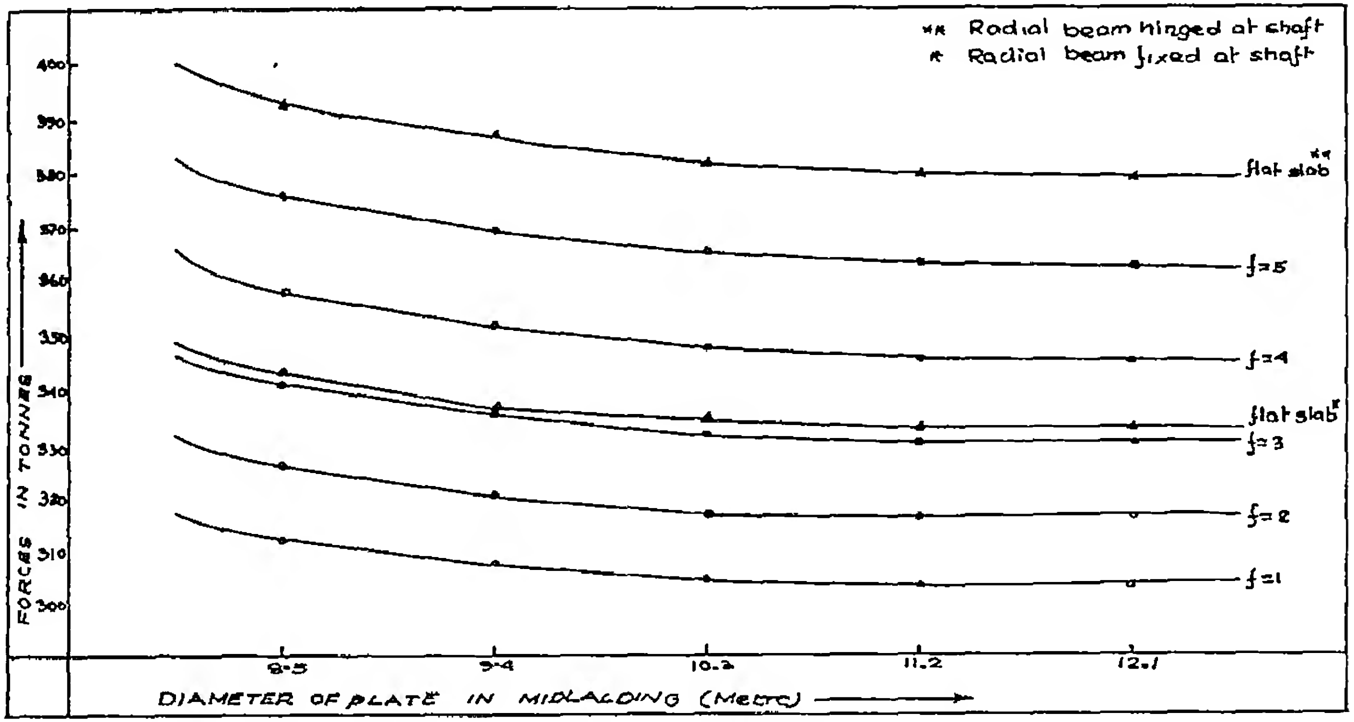


Fig.-20A · AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOME
(NUMBER OF COLUMNS - 8)

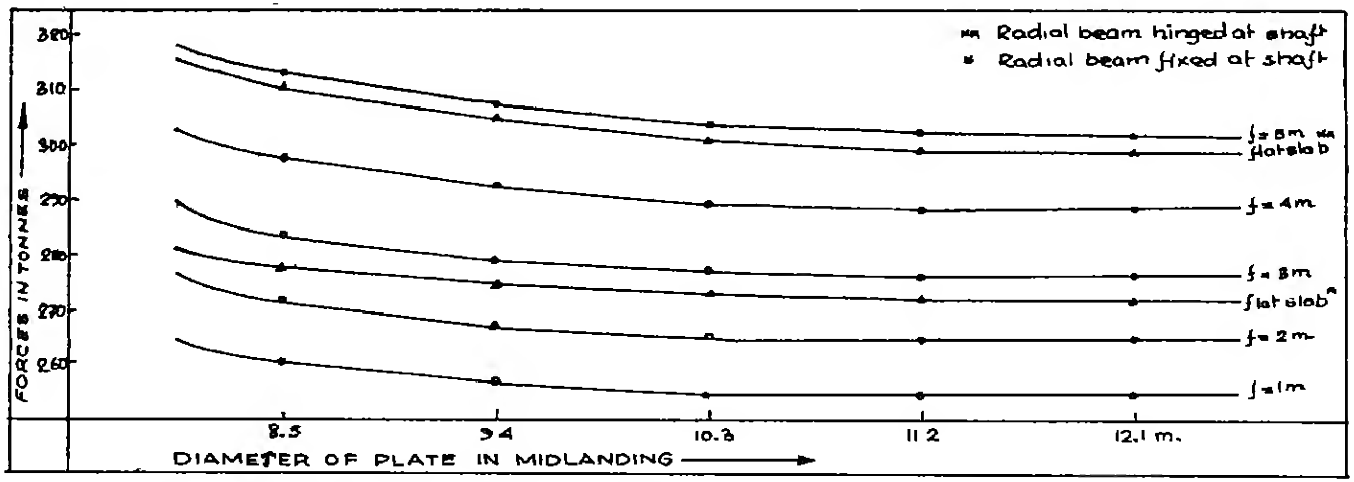


Fig.- : AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS-10)

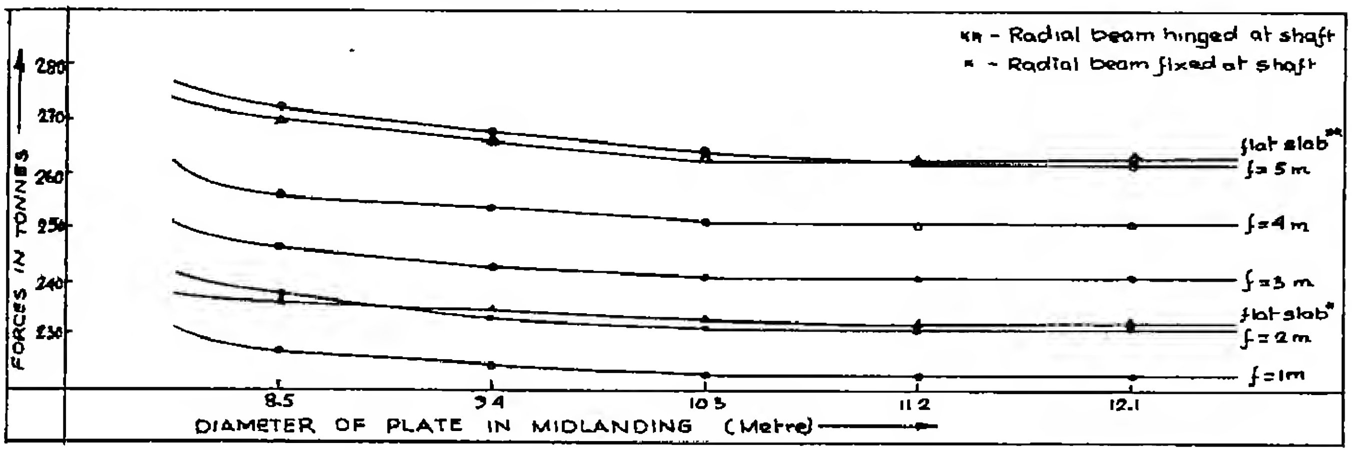


Fig.- 26B : AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS -12)

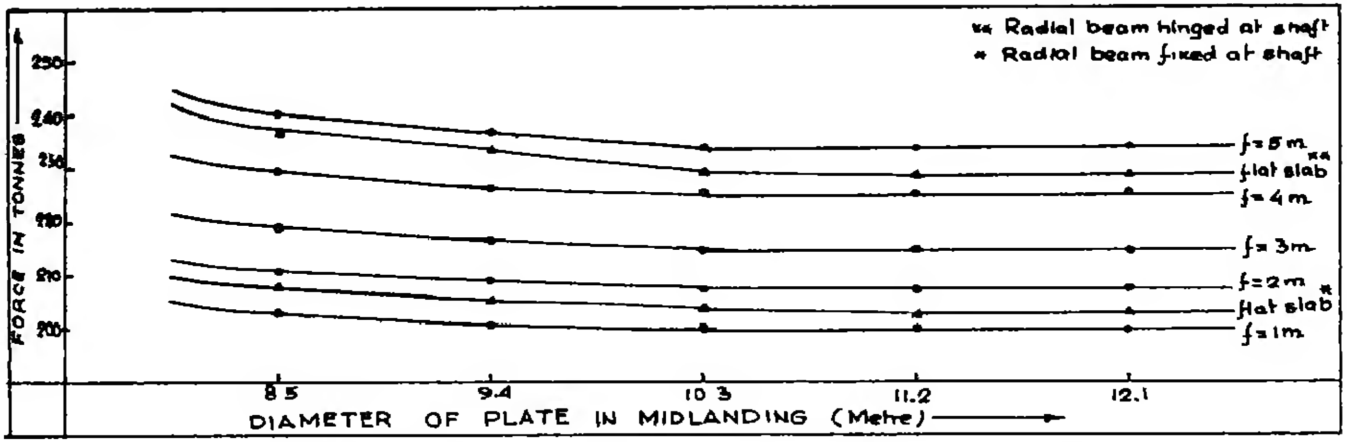


Fig- : AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS-14)

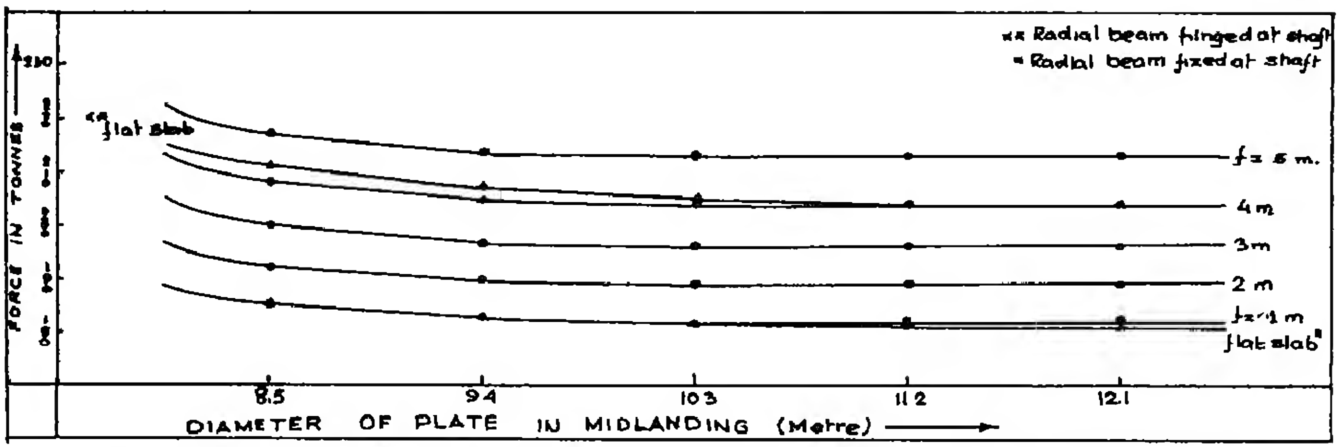


Fig.-20C: AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS 16)

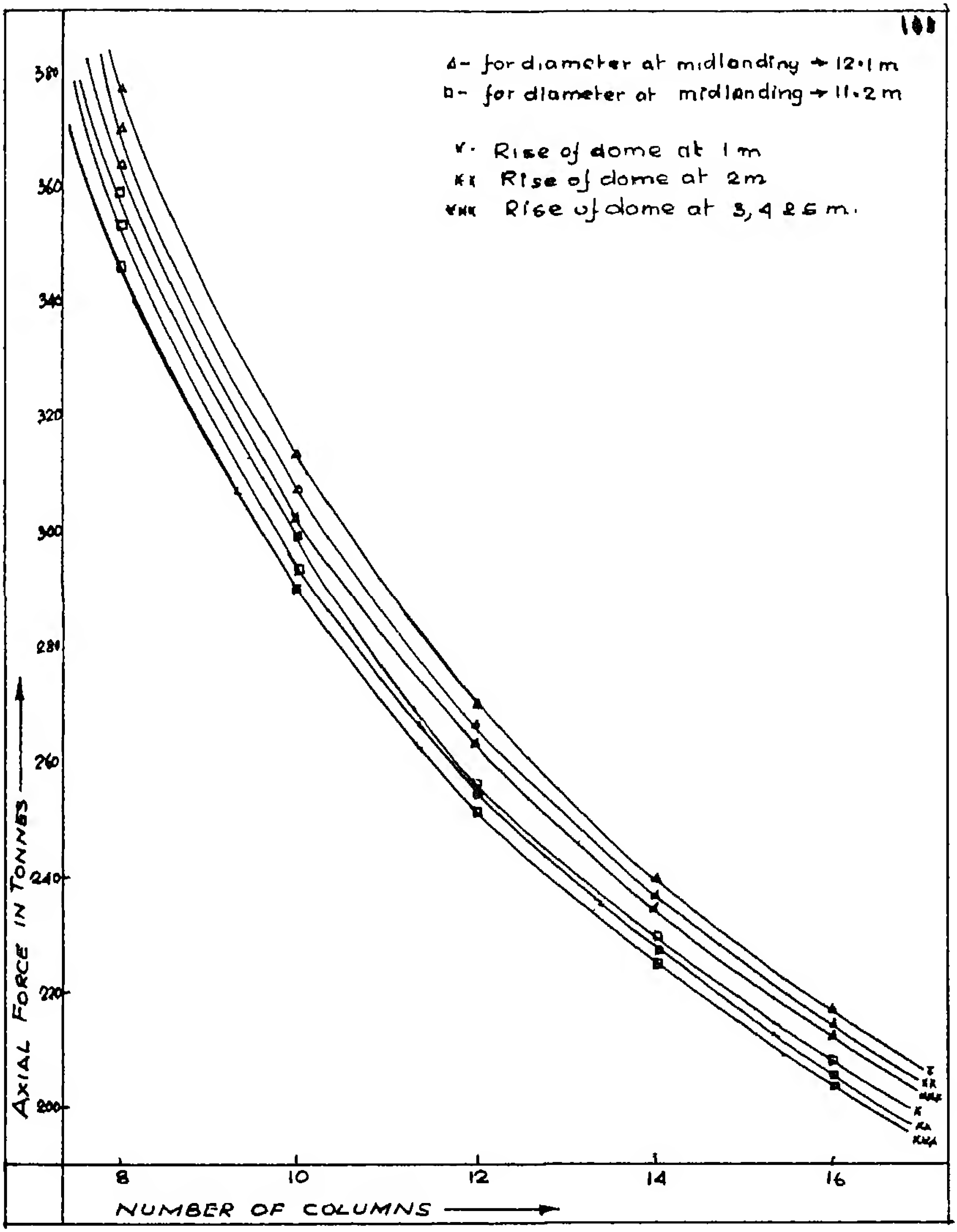


Fig.- 20D : AXIAL FORCES IN COLUMNS

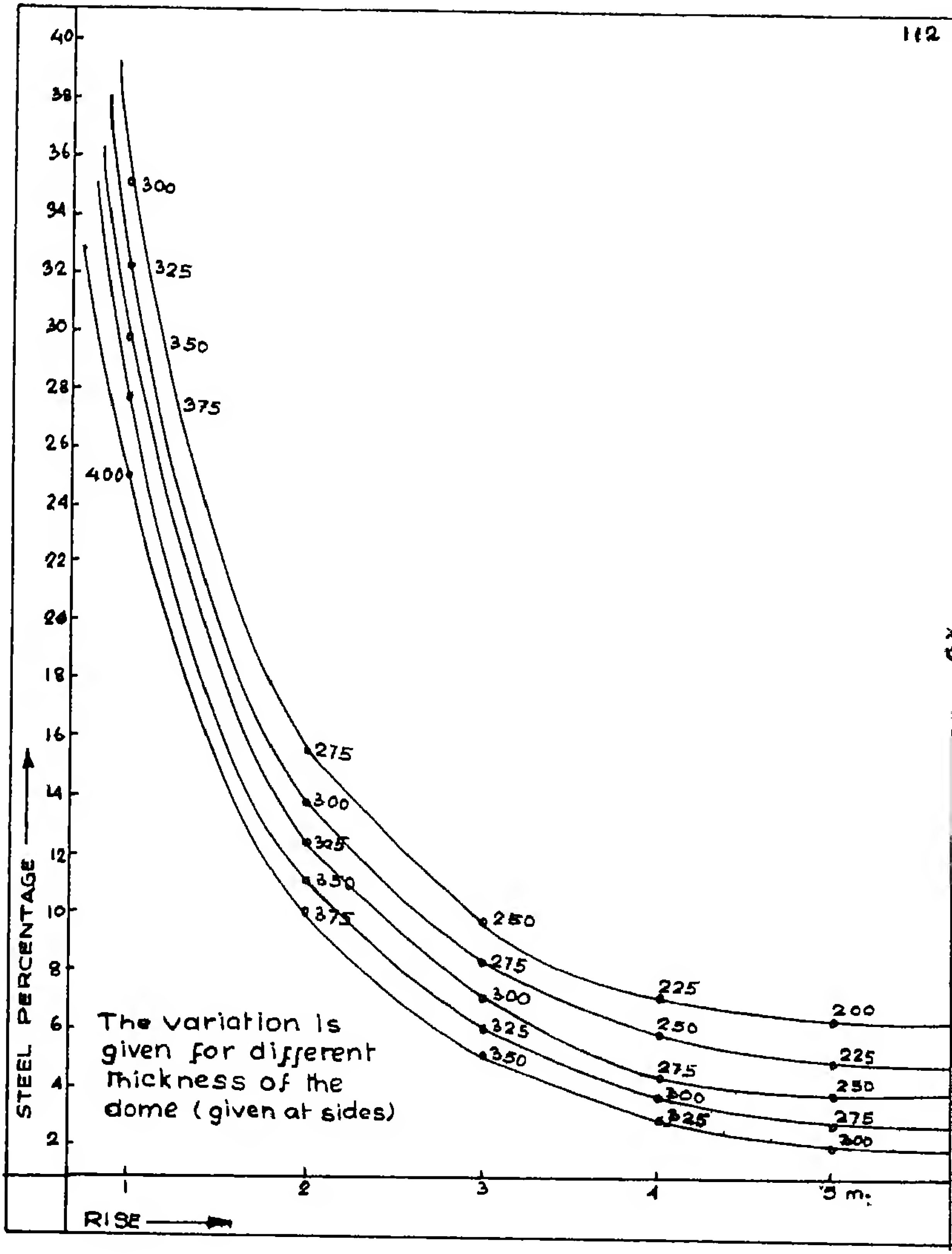


Fig.- 21 : PERCENTAGE OF STEEL IN BOTTOM DOME w.r.t. RISE OF DOME

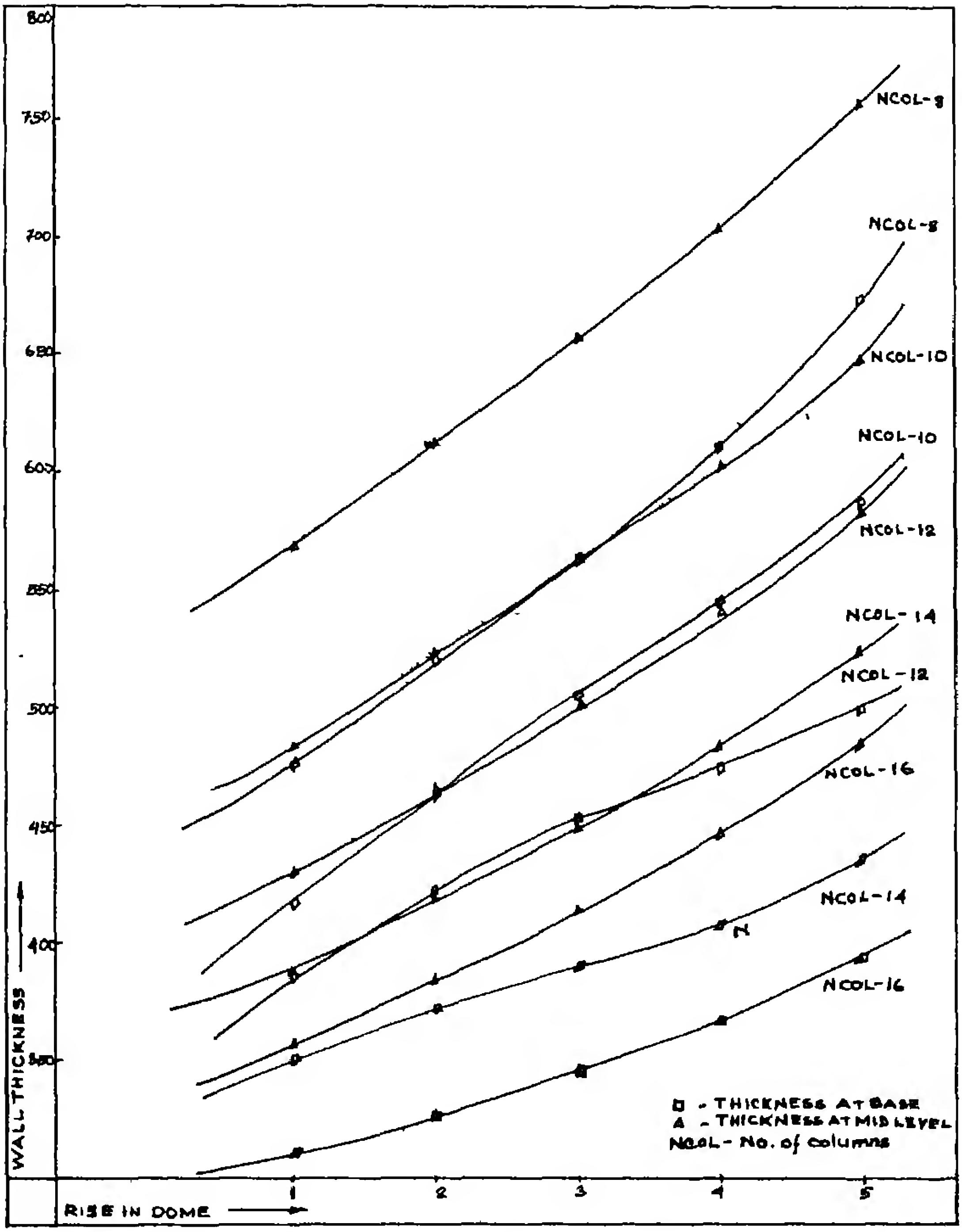


FIG - 22 _ VARIATION OF TANK WALL THICKNESS W.R.T. RISE OF DOME

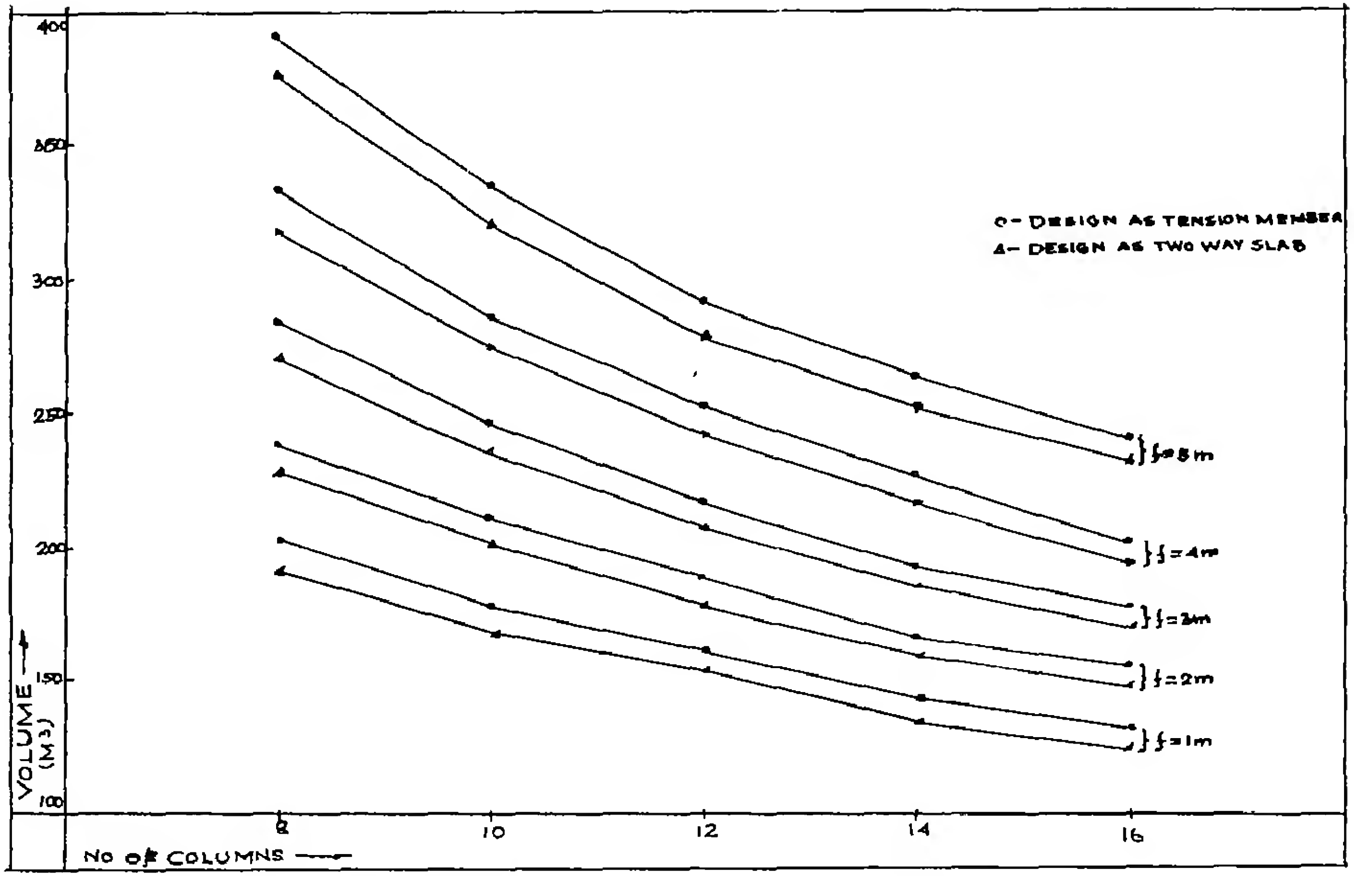


Fig – 23 Volume of Tank Wall for Different Ribs in Bottom Dome

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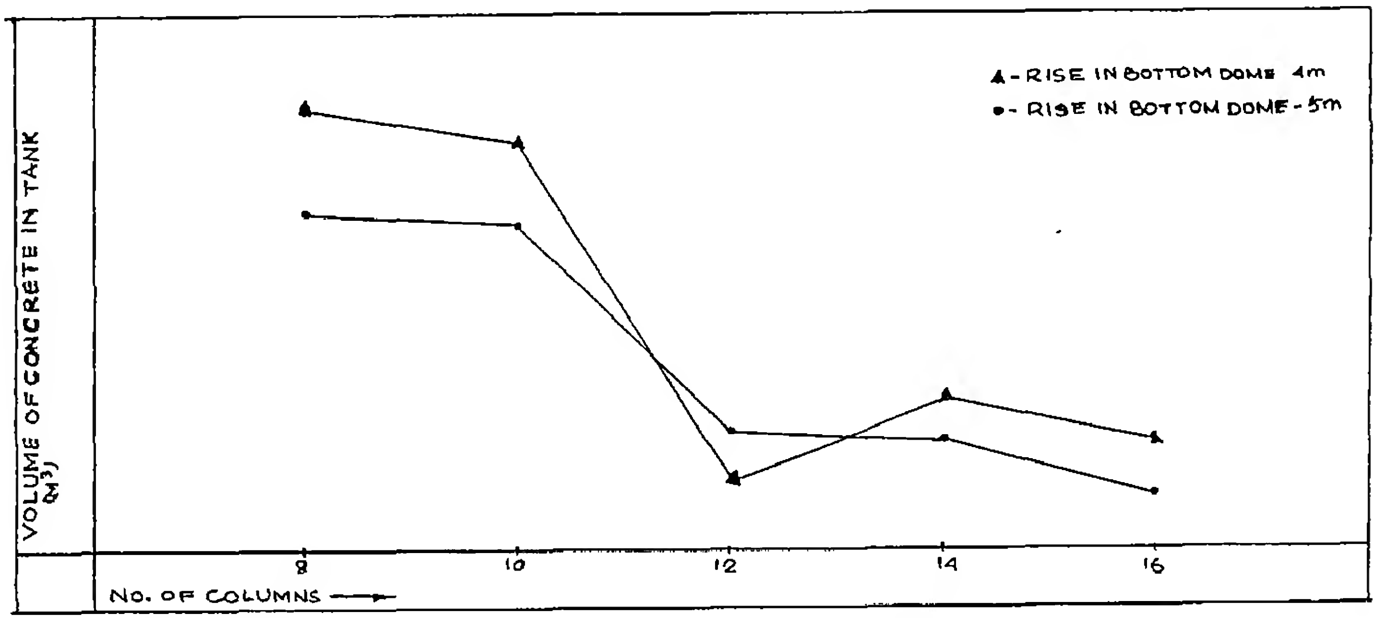


Fig.- 24 TOTAL VOLUME OF CONCRETE FOR DIFFERENT STAGING

OPTIMIZATION OF SHAPE OF WATER TANKS

5.1 INTRODUCTION

In designing a structure, the engineer has a choice of two basic strategies.

The traditional design procedures consists of first adopting intuitively the geometry and materials of the structures and then calculating for specified design loads the value of behavioral or state variables such as stresses, strains and displacements. After successive modification of the geometry, the procedure is then repeated until the calculated behavior satisfies certain prescribed requirements which are usually expressed in the form of inequalities representing upper limits on stresses and displacements or a lower limit of load capacity. The disadvantages of the foregoing procedure are obvious. First much computational effort may be wasted on successive analysis. Second, the design can be highly uneconomical, even when an intuitively selected solution satisfied the behavioral constraints.

To eliminate these drawbacks, the traditional procedure may be somewhat reversed. In what is termed optimal design first the required structural behavior together with design loads and geometrical constraints are specified and a further quantity termed cost or objective function is also defined. The aim of the subsequent computational effort is to select the geometry and possibly materials of the

tructure, so that the required behavior is achieved at mallest possible cost.

5.2 PREVIOUS STUDIES

A shape optimization process begins by the definition of the geometry, its parameter & limitation and their lependencies on the design variables. The design of water tower shape depends on the design of cylindrical shells, conical shells, spherical domes and circular beams. To have an optimized shape of water towers all the components should be designed in such a way so that it satisfies the objective function. Here the is defined by the line of the different shells viz., cylindrical, conical, spherical joined at the points where the ring beams are holding them together.

Marcelin & Trompette / 8 / studied about optimal shape of thin axisymmetric shells. Their main aim was to obtain shapes giving rise to uniform maximum reference stress s along the variable boundaries. The unconstrained associated minimization is solved by the variable method of Davidon-Fletcher-Powel with the help of a finite element model. And they tested the results on various examples viz., shape of a drop of water, shape of a bottle, etc.

Kapoor & Hariharan / 4 /studied about the optimal design of reinforced concrete chimneys with reference to a practical design and showed the amount of savings that could ive been made.

Thambiratnam et al / 14 / recommended about minimum weight design of cylindrical wall tanks. They recommended height to diameter ratio of cylindrical water tanks for smaller capacities.

Melchers & Rozvany / 9 / considered about the optimum lower bound design of reinforced tanks. They considered circular cylindrical shell subjected to outward radial pressure without any axial (longitudinal) loading for two cases e.g., free at top & fixed at base and free at top & hinged at base. They showed that designs based on the lower bound theory the regions and region boundaries gave total lower steel volumes than equivalent design based on elastic stress distribution, or an uniform isotropic yield line analysis.

Thambiratnam et al / 15 / studied about the maximization of maximization of natural frequencies of cylindrical shells. They recommended for designing of a structure to isolate from destructive effects of resonant vibration rather than strenghtening it to withstand the vibration. This is achieved by elevating the natural frequencies of the structure so that they are sufficiently higher than the frequencies of all possible dynamic excitations in the vicinity. They showed that the lowest frequency a for axisymmetric vibration kept on increasing with the number of slopes in the tank wall and limiting case was a shell whose outer wall was curve in the form of a circular arc with zero thickness at the top.

Thompson & Hunt / 17 / made a laboratory study about effect of asymmetry of an element if the element is being designed as symmetrical member. They observed that the load bearing capacity of elastic arches, cylindrical shells, complete spherical sections, complete cylindrical sections reduces with the degree of asymmetry and the that follow a curve of a hyperbola.

The problem of optimum design is defined as the determination of a design specified within a set of parameters, which minimizes certain structural property. The admissible designs from which the optimal has to chosen is called the class of safe designs.

5.3 FORMULATION OF MODEL

The primary shape of the model is taken to be a circular intze tank (Fig.-). Having the primary shape in mind here six other cases are considered by giving some preassigned values to some of the variables.

The variables are listed as below:

1) Set A

a) Height of the first section : H2

b) Height of the second section : H3

c) Radius of the top ring beam : R1

d) Radius of the middle ring beam : R2

e) Radius of the bottom ring beam : R3

2) Set B

a) Radius of the shaft : R4

b) Outer radius of the plate over shaft : R5

c) Thickness of the plate : T6

3) Set C

a) Height of top dome : H1

b) Height of bottom dome : H4

4) Set D

a) Thickness of the top dome : T1

b) Thickness of the top conical section : T2

c) Thickness of the bottom conical section : T3

d) Thickness of the bottom dome : T4

e) Thickness of shaft

The objective function here is to obtain a shape of the tank for a particular capacity so that it transfer least weight to its staging.

Five basic design variables (Set : A) have been considered here. Out of these variables height of the first conical shell is calculated from the volume relationship using the other four variables.

Values of three variables (Set : B) have been kept fixed depending on the type of tank.

Rise of the spherical domes have always been kept fixed at one sixth of the radius of the dome at base.

Thickness of the sections have been varied within the prescribed maximum and minimum values. The reinforcement is also restricted by a maximum (0.035) and a minimum value / 26 /.

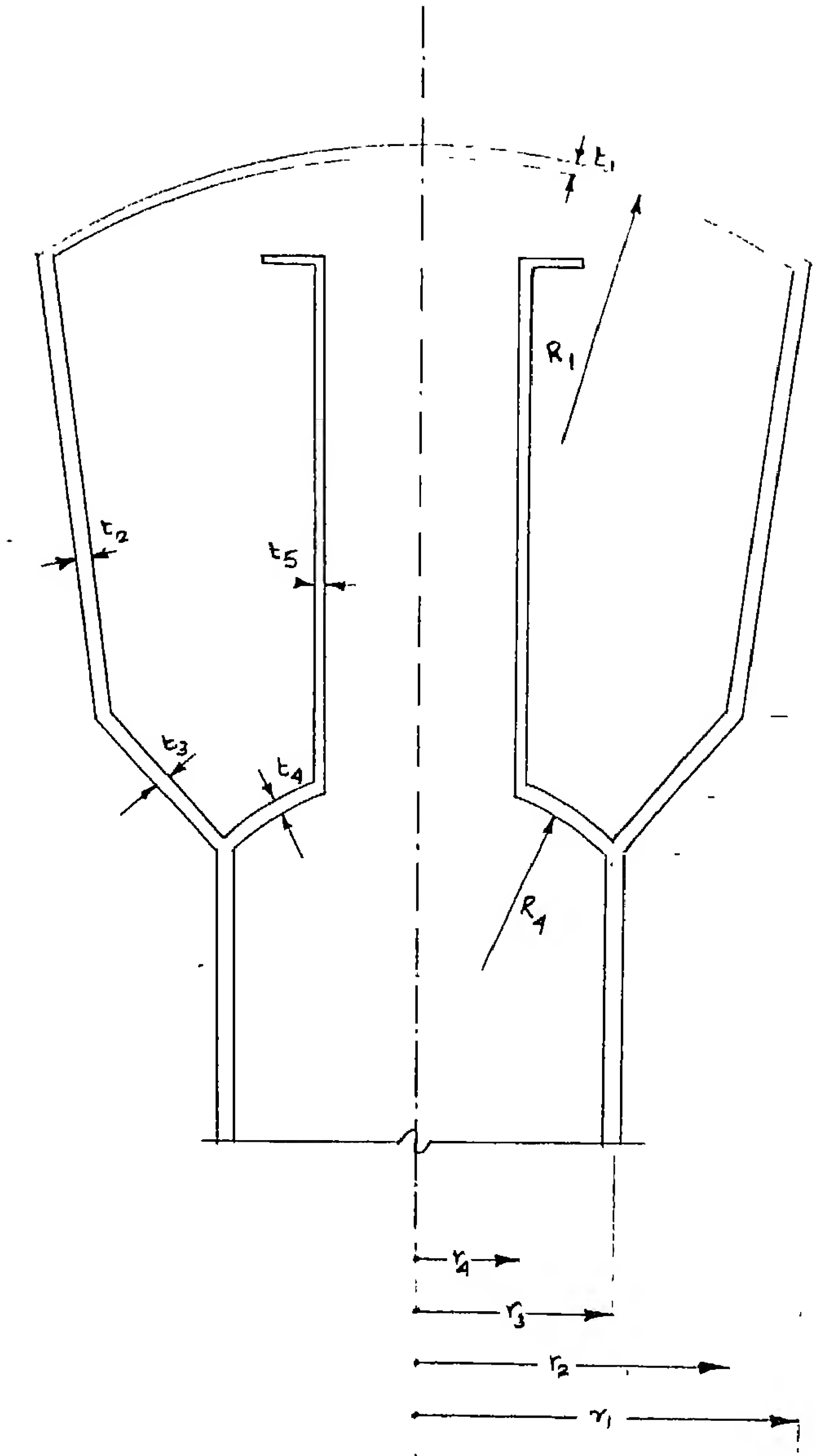
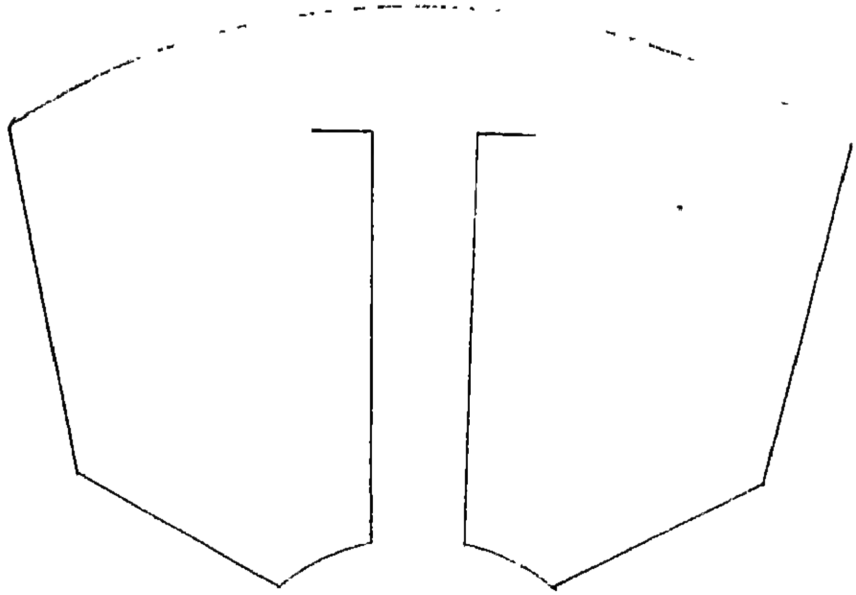
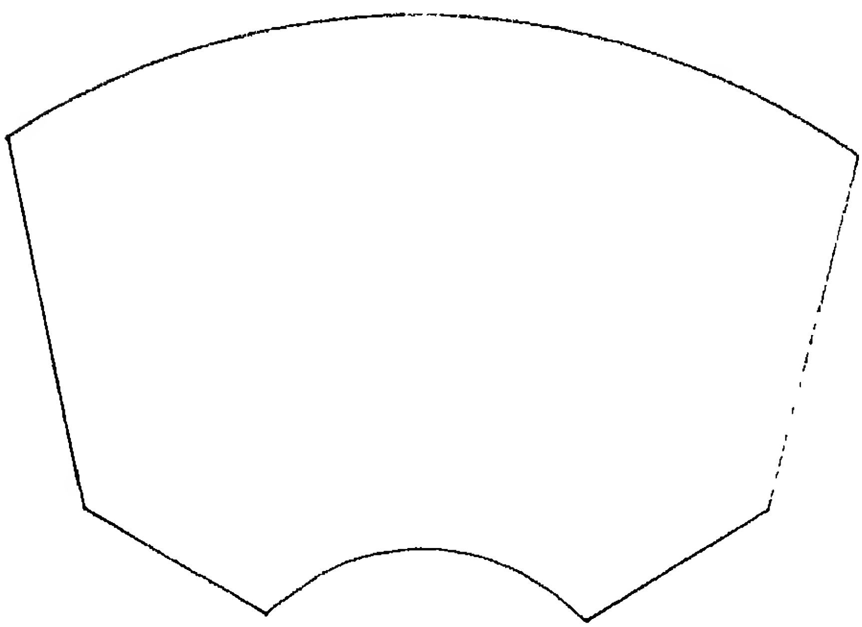


Fig.-25 SECTION OF A TYPICAL INTZE TANK

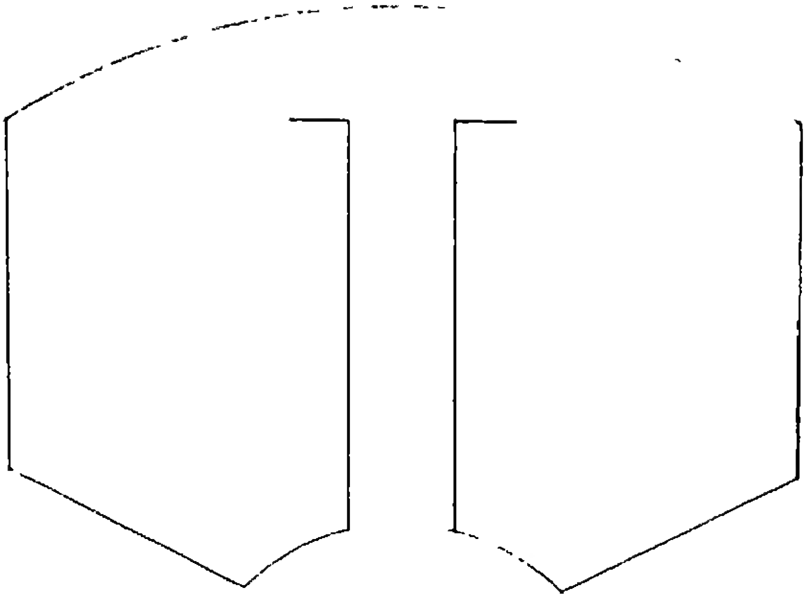


Case-1 : All values are varying

Fig-26A - SECTION FOR OPTIMIZATION

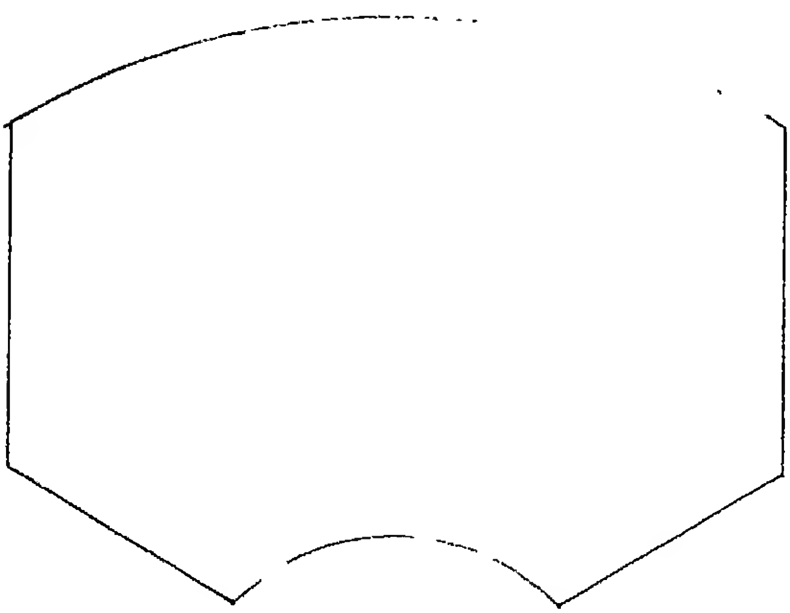


Case-4: R4=0; R5=0

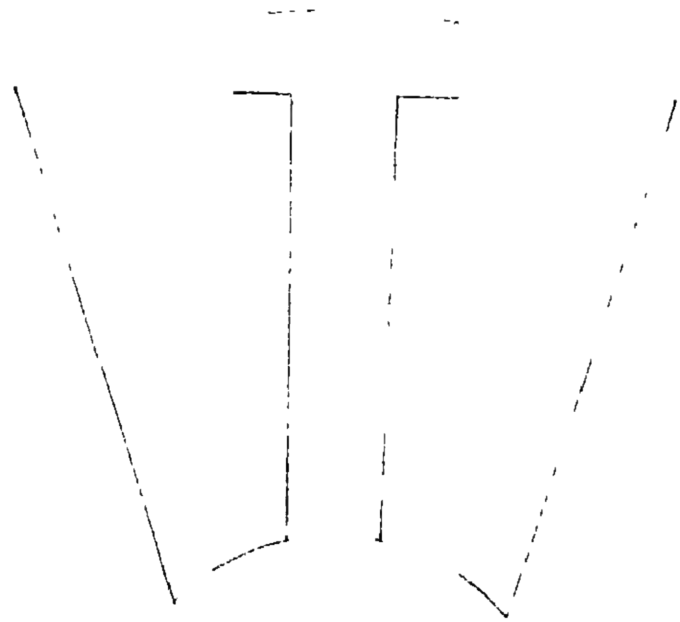


Case-2 : $R2 = R_1$

Fig. 26C - SECTION FOR OPTIMIZATION

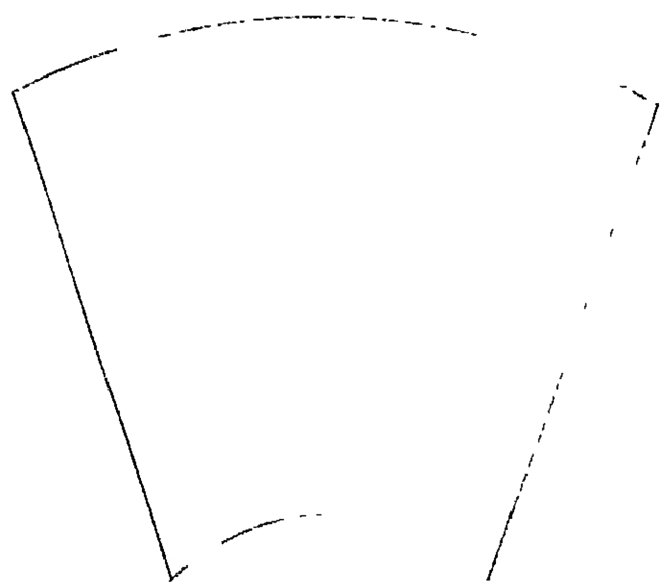


Case-5 : $R2 = R1$, $R4 = 0$, $R5 = 0$



Case - 3 : H3 = 0, R3 = R2

Fig. -26E SECTION FOR OPTIMIZATION



Case - 6 : R3 = R2, H3 = 0

Fig. -26F SECTION FOR OPTIMIZATION

Case-7: R3 = R1, R2 = R1
H3 = 0, R4 = 0, R5 = 0

Fig. 26G – SECTION FOR OPTIMIZATION

5.4 DERIVATION OF ALGORITHM

The problem here has been taken as a non-linear real value programming problem. The tank has been designed sequentially from top to bottom. The weight of each part has been calculated at every stage which is added finally to have the total weight. During design procedure the values of the design parameters are regularly modified to have a realistic value in terms of construction at field.

The constraints in this optimization are mainly thicknesses and the percentages of reinforcements in the sections. he design variables for the optimization process are listed below for different cases. The table shows whether any variable have a fixed value, zero value or value equal to that of another variable.

TABLE 8 - LIST OF PARAMETERS IN SHAPE OPTIMIZATION

Variables :	R1	R2	R3	R4	R5	H2	H3
Case - 1 ::	V	V	V	C	C	V	V
Case - 2 ::	V	R1	V	C	C	V	V
Case - 3 ::	V	V	R2	C	C	V	0
Case - 4 ::	V	V	V	0	0	V	V
Case - 5 ::	V	R1	V	0	0	V	V
Case - 6 ::	V	V	R2	0	0	V	0
Case - 7 ::	V	R1	R1	0	0	V	0

V : Variable parameter, C : Fixed parameter

Though this is a constrained optimization problem, it has been converted to unconstrained optimization problem by incorporating the thickness and reinforcement constraints during the designs of the sections. For this the following algorithm has been used.

1) Start with an intermediate value of thickness and reinforcement.

2) Call subroutine for section design.

3) If T > Tmax return to the main subroutine and changes the design variables and start the procedure again, else go to the next section.

4) If Pst > Pmin go to step (5) else go to step (6).

5) If T > Tmin reduce T and go to step (2) else write T = Tmin, Pst = Pmin and go back to the main subroutine.

6) If Pst > Pmax increase T and go to step (2) else go to the next step.

7) If T > Tmin write T & Pst and go back else write T = Tmin & Pst and go back to the main subroutine.

For this unconstrained optimization problem "Direct Search Method" has been used. This is because, it does require derivative of the objective function. It will be a difficult job to have derivative for this complicated problem if not impossible because it deals with a number of sections. For the optimization procedure "Pattern Search Method" have been following the "Hookes & Jeeves" method / 10 /.

5.5 STUDY OF RESULTS

For an arbitrary value of input the programme calculates diameters and the height of of the sections. For absurd values the number of iterations will be more otherwise it calculates the minimum weight within 20 iterations.

Another problem that developed was if the angle of the second conical section is not being restricted then sometimes the radius R2 becomes less than radius R3.

The results of the programme suggests that : 1) upto a capacity of 400 KL tanks of case#7 may be used and above this value the section becomes uneconomical, 2) above this and upto a capacity of 900 KL tanks of case#6 and case#3 may be used, 3) above this other cases should be used.

In case of a different R1 and R2 value in case of case#1 & case#4 the R1 and R2 value becomes nearly equal after a seven or eight iterations.

The weight of concrete to weight of water ratio varied between 0.86 and 1.03.

5.6 CONCLUSION

In engineering design two basic aspects are optimization and aesthetics, where "optimization" will consider the problem of safety serviceability etc. and "aesthetics" takes care of the artistic value of the structure.

The aesthetic rationale of design is to have a well balanced & harmonious section of a structure with a sense of beauty / 17 /. The design should have a symbolic image also. It should represent the motif of historical background, traditional culture. It should be environmentally compatible with the natural features of the location.

But when the design comes to a designer, he or she must use the common sense for the design and not merely follow some rules.

5.7 SCOPE OF FUTURE STUDY

1) A dynamic programming may be made for the whole structure for weight minimization cosidering a tower supported on a central stem.

2) A Cost optimization programme may be developed for water tower design.

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